PROCEEDINGS
THE INSTITUTION OF
CIVIL ENGINEERS

PART I
JANUARY 1952

LIBRARY

ORDINARY MEETING

6 November, 1951

WILLIAM HENRY GLANVILLE, C.B.E., D.Sc. (Eng.), Ph.D.,
the retiring President, in the Chair

The President said that it was with great regret that he had to inform the Meeting of the death, during the Institution recess, of Mr Guy Howard Humphreys, Member of Council. A resolution of condolence had been passed by the Council and conveyed to the members of Mr Humphrey's family.

The Council reported that they had recently transferred to the class of

Members

William Duncan Arnot, B.Sc. (*St. Andrews*).
William Alexander Ronald Baker, B.Sc. (Eng.) (*Lond.*).
Edward Hugh Bateman, M.A. (*Cantab.*), B.Sc. (Eng.) (*Lond.*).
Herbert William Beasley.
Mark Bernstein, B.Sc. (*Cape Town*).
Leonard Harry Bird, J.P.
John Alexander Buchanan, B.Sc. (*Edin.*).
Robert Carey.
William Dempster.
Paul Thomas Fletcher, B.Sc. (Eng.) (*Lond.*).
Robert Drake Hawkins, B.Sc. (*Bristol*).
Alan Cleveland Hobbs, B.Sc. (Eng.) (*Lond.*).
Kenneth Palmer Humpidge, B.Sc. (*Bristol*).
Arthur Ernest Jennens, B.Sc. (*Birmingham*).
Vivian Dottin Joll, B.Sc. (*Bristol*).
Christopher Wallis King.
Thomas Trajan Lambe, B.Sc. (Eng.) (*Lond.*).
Charles Kenneth Larard, B.Sc. (Eng.) (*Lond.*).
William Bruce Lawson.
Kenneth Douglas Mathewson, B.A. (*Cantab.*).
Dudley Holt New, B.Sc. (Eng.) (*Lond.*).
John Albert Newell.
David George Norton, B.A. (*Cantab.*).
William John Norton, B.A. (*Cantab.*).
Terence Patrick O'Sullivan, Ph.D., B.Sc. (Eng.) (*Lond.*).
John Wyngate Perkins, B.A. (*Cantab.*).
Donald Andrew Gladstone Reid, B.Sc. (*Glas.*).
Gerald Warburton Rooke, B.A., B.A.I. (*Dublin*).
Horace Cuthbert Rowley.
Norman George Simpson.
William James Sinclair.
Kenneth Gordon Stevens.

John Robertson Stewart, M.B.E., B.Sc. (*Witwatersrand.*).
Frank Dennis Thomas, B.Eng. (*Liverpool*).
Noel Edmund Viner Viner-Brady.
Walter Alfred Watts, B.Sc. (Eng.) (*Lond.*).
Richard Cecil Whitehead, B.Sc. (*Birmingham*).
Rees John Williams.
Edgar Burke Wilson, B.Sc. (*Belfast*).

and had admitted as

Students

Hari Aggarwal.
Inder Saran Aggarwal.
Adelwin Tajmil Abdue.
William Ronald Agnew.
James Aikman.
Anand Ganesh Akerkar, B.E. (*Bombay*).
John Michael Alty.
John Michael Douglas Anderson.
William Greer Anderson, B.A., B.A.I. (*Dublin*).
John Frederick William André.
Brian Archer, B.Sc. (*Durham*).
Paul Rex Argue.
David John Armstrong, B.A. (*Cantab.*).
Raymond Arrowsmith.
Senagi Attygalle.
John Headland Ayers.
Gerald Wallace Bailey.
Gordon Leonard Bailey.
David Richard Banham.
Robert Edward Barnard.
Eric Lionel Barron, B.Sc. (Eng.) (*Lond.*).
Henry Grey Barter, B.E. (*New Zealand*).
Alexander Ogilvie Beattie.
Geoffrey Beaumont.
Ross Beaumont.
John Colin Beevers.
Douglas Bell.
Guy Davis Bell.
Donovan Harry Bennett.
Henry George Bentley.
Edward Faux Best, B.Sc. (*Nott.*).
David William Betts.
John Bishop.
George Allan Blacker.
Arnold Blades.
John Clifford Blanchard, B.Sc. (Eng.) (*Lond.*).
Narayan Ganesh Bondre.
Desmond Frederick Boon.
John Patrick Maurice Booth.
Sailendra Nath Bose, B.E. (*Calcutta*).
Joseph John Bradley.
John Walter Bradstock, B.E. (*New Zealand*).
John Franklyn Brayne.
Alec Thomas Brown, B.Eng. (*Liverpool*).
Edward Alan Harvey Brown.
David Michael Bunn.
William Spiers Burnside.
Kenneth Gordon Burton.
Richard Edwin Vivian Butler, B.A. (*Oxon.*).
Stephen William Henry Butler.
Humphrey Richard Ian Cannon.
William Richard Carmichael.
Peter Carr.
David Derek Casstles.
Richard Gordon Cecil.
Peter George Chapple.
George Surachat Chatai.
Witold-Jozef Chodzko-Zajko.
Trevor Kenneth Clarkson.
Gerrard Close.
Stanley Robert Cochrane, M.Sc. (*Belfast*).
Eric Maxwell Cole.
Peter Stanley Coleman.
Peter William Cooke.
Alan John Cooper.
Graeme Robin Cortvriend.
William Frank Cotton.
Colin Alfred Cowie.
David Dingley Cox.
Edward Gray Cran, B.Sc. (*Aberdeen*).
Frederick Harry Craven.
John Horace Hugo Creed.
Dennis John Victor Crocker.
Malcolm Sidney Cuckow.
Ian Ronald Cumberworth.
William Mackie Currie.
Herbert Lewis Curzon, M.A. (*Cantab.*).
Clifford John Darke.
Christopher John Davies.
David Maurice Davies.
Haydn Keith Davies.
John Davies.
Maurice William Davies.
Peter William Davies.
Patrick James Alec Davison.
Ronald Colin Deacon.
Anthony John Dear.
Emil Hugh de Quervain.

Eustaco Gerald de Zilwa.
Ralph Selwyn Dixon.
John Robert Tomkys Douglas, B.Sc. (*Birmingham*).
Daniel Anthony Patrick Downes.
Peter Royston Drysdale.
Kenneth Henry George Duce.
Daniel Joseph Duckett, B.E. (*National*).
Derek Albert Duly.
Nigel Kenneth Duncan.
Raymond Duncan.
James Dunsmore.
Michael Arthur Allen Dyer, B.Sc. (*Witwatersrand*).
Michael Charles Earwaker.
William Stewart Easson.
Harry Dean Edmondson, B.Sc. (Eng.) (*Lond.*).
Paul Harper Dickinson Edwards.
John Michael Edbrooke.
Alfred William Egan.
William Denis Elders.
Colin Victor Caldwell Elliott.
Malcolm Lyall Learmonth Ellis, B.Sc. (*St. Andrews*).
Charles John English.
Aneurin John Evans, B.Sc. (*Wales*).
Chalmers Henry Fairweather.
Peter Farrington.
Derek Bernard Fawcett.
Michael John Feazey.
Merennege Ratnapala Fernando, B.Sc. (Eng.) (*Lond.*).
Rodney Arthur Fisher.
Adrian Leo Fishwick.
Patrick Alan Fitzpatrick, B.Sc. (Eng.) (*Lond.*).
Eric Bryan Flesher.
Geoffrey Buchanan Fort.
Edward Francois.
Frederick John Thomas Fraser.
Thomas Frederick Frewin.
Robert Singleton Furness.
Ian Gouch Gibbons.
Kenneth Giddings.
Allan Charles Gill.
Ian Goss-Ross.
George Rupert Good.
George Evan Graham.
Peter Douglas Gray.
Desmond William Burbridge Green.
Trevor Rex Green.
Alwyn Griffiths.
Anthony Gross.
Hathamunage Don Tudor Davy Nimal Tissa Gunawardhana.
Lionel Peter Haines.
Peter Hanby-Holmes.
Virender Kumar Handa, B.Sc. (*Calcutta*).
David James Hannay.
Kenneth George Hatton.
John Henry William Havelock.
Charles Anthony Hawkins, B.Sc. (*Belfast*).
Douglas Oswald Hay.
Derek John Norman Hele, B.Sc. (Eng.) (*Lond.*).
John Russell Henderson, B.A. (*Dublin*).
Leslie Nathan Henderson.
James Heron.
John Desmond Heyworth.
Arthur Bradley Hill.
Roger Paul Hills.
Brian Patrick Lewis Hitchen.
Peter John Holdcroft.
Roy Holdman.
Michael Holley.
Geoffrey Boscawen Howard, B.Sc. (Eng.) (*Lond.*).
Peter John Howard.
Jack Clifford Howe, B.Sc. (Eng.) (*Lond.*).
Brian Hubbard.
Alan Maurice Hunt.
Harold Bowler Hunt.
Hugh Peter Hunt.
Garden (Garry) Witten Ingram.
Donald Arthur Jackson.
Ian Walter Dunlop Jackson.
Kenneth Raven Jackson, B.Eng. (Sheffield).
Patrick Jennings.
David Geoffrey Jewell.
Geoffrey Max Johnson.
Robert Jolly.
Aravinda Joshi.
Nadarajah Kanthasamy.
Peter Henry Norris Kendall.
Hugh Kermode.
Geoffrey Brian Kerr.
John David Kerr.
William David Ross Kerr.
Gordon William King.
William Langford.
Robert Weir Lees.
Harvey Leeson.
Ivan Charles Levy.
Barry Levy, B.A. (*Cantab.*).
David Llewellyn.
John Godwin Longbottom.
John Frederick Lowe.
William Wallace Lowson.
Ralph Edward Luke, B.Sc. (Eng.) (*Lond.*).
Michael Richard Thomas Mabbs.
Pearse Mary MacLochlainn, B.Sc. (*Belfast*).
Kenneth Relton Madsen.
Peter James Alexander McBean.
Dermot William McCaffrey, B.A., B.A.I. (*Dublin*).

Jack Kinnell McDade.
Howard Douglas McGarr.
John McKenna.
John William Bernard McKenna.
Thompson Joseph McKie.
James Lyons McKinnon.
Richard Donald McLaren
David Cook McLean.
Ronald Murdoch McLeay.
Colin Gordon McMillan.
John Morton McNaught.
Robert Bryan Howard Malyon.
Otto Heinz Mankarz.
Michael Charles Mann.
Edwin Mark.
Robert Marshall.
Bryan Grundy Martin.
Peter James Mason.
Michael John Masterson.
John May.
Brian Mayfield.
John Brian Meers.
John Haydon Mead.
Edward Melling.
Ewald Meyer.
Kenneth James Midson.
Stanley Anthony Miles.
Anthony Maurice Millership, B.Sc. (Eng.) (*Lond.*).
Alan David George Miles.
Roy Eric Mills.
John Milsom.
David Berchmans Minch, B.A.I. (*Dublin*).
John Frederick Moore.
John Herbert Moores.
Peter David Morris.
John Munro.
John Bloxham Murray.
Joseph Jegasothy Mylvaganam.
Nagendram Nadarasa.
David Anthony Neale.
George Howard Newton.
Trevor Allan Newton.
Allan Dennis Nicholls.
Richard Rhodes Nicholson.
Robert Joseph Nisbet.
Don Bermulan Prematilaka Nissanga.
Derek Ormerod.
Derek Owen.
William Trevor Owen.
Michael Hugh Ward Page.
John Edward Palmer.
Herbert Bridge Parkin.
Maurice Parks.
Thomas Victor Parrott.
Anthony Sydney Pateman.
Walter Sutter Paterson.
John Firman Payne.
Robert Charles Peak.
John Henderson Percy.
Alan George Pollock.
David Cyril Rodney Poore.
Kenneth John Prangnell, B.Sc. (Eng.) (*Lond.*).
Douglas William Prentice.
Geoffrey Prior.
Charles Henry Pye.
Anthony John Pyke.
Ponniah Rajendra.
Andrew Gilbert Reid.
David Clifford Reynolds.
David Blake Rich.
Robert Douglas Riddett.
Brian Mayson Rimmer.
Duncan McRae Robertson.
Anthony John Metcalf Robinson.
Brian Anthony Rogers.
Paul Briant Rooksby.
Kenneth Hugh Ross.
Keith Murray Rowe.
David Minton Russell.
Alan John Saines.
Sujit Kumar Sarkar, B.E. (*Calcutta*).
James Clayton Scarisbrick.
Andrew Noel Schofield.
Lewis Arnold Schofield, B.Sc. (*Leeds*).
David Alexander Noel Scott.
Charles O'Malley Haskins Seale, B.A., B.A.I. (*Dublin*).
Harold Francis Seed.
Abeyaratne Bandara Ehelepola Seneviratne.
William Musgrave Sewell, B.A. (*Cantab.*).
Edward Francis Shields.
Harry Ratcliffe Shotton, B.Sc. (*Durham*).
Alan Simpson.
John Simpson.
Donald Anthony Sinclair.
Clifford Anton Skerl.
Christopher Edward Slade.
George Sloan.
Robert William Smith.
Sydney Herbert Smith.
Henry Graham Stockham.
John James Steele.
Peter Henry Stewart.
Brian Arthur Stevens.
Derek Cecil Stringfellow.
Duraipillay Subramani, B.E. (*Madras*).
Maurice James Sutherland.
Allen Fauconnier Sutton.
Lucius Frederick Charles Tarrant, B.A., B.A.I. (*Dublin*).
Eric Peter Ringquist Tatman.
Andrew Paton Taylor, B.Sc. (*Glasgow*).
John Taylor, B.A., B.A.I. (*Dublin*).
Kenneth John Teasdale.
Crispin Felix Thompson, B.Sc. (*Rangoon*).
John Peter McKay Thomson.
Alan John Tolbutt.

Patrick Thomas Trail.
John William Turnbull, B.E. (*New Zealand*).
Harold Peter Turrell.
Eric Victor William Turton.
Vernon Emmanuel Hector Vallun.
Paul Vavasour.
Denis Vipond, B.Sc. (*Manchester*).
Alan Walker.
Geoffrey Walker.
Eric John Walshe, B.A., B.A.I. (*Dublin*).
William Wands.
Kenneth Michael Wardle.
John Arthur Watson.
John Roland Watson.
Nigel Anthony Watson.
Thomas William Weaver.
Gerald Robert Webley.
David Munro Weir.
Francis Michael Whitehead.
Sathiavan Wignaraja.
Francis Clive Williams.
David Williamson.
Keith Francis Denis Wilson, B.Sc. (Eng.) (*Lond.*).
Patrick Woolcott.
George Arthur Wright.
Robert Anthony Yates.
Alexander Nicol Young.
Mohammed Yusuf.

The Secretary announced the awards which had been made by the Council for Papers presented during the Session 1950–51. Details of these awards are given on p. 25.

The President presented Coopers Hill War Memorial Medals to Mr E. O. Measor, Member, and Mr D. H. New, Member, and the Institution Medal to Mr J. H. Cuckney, Student.

The Retiring President said that it was now his pleasant task to introduce the new President, although he was quite sure that no introduction whatever was needed. Mr Quartermaine had played a leading part in the Institution activities for many years. He had done a great deal for the Institution, and a great debt of gratitude was already due to him. Dr Glanville was quite sure that when Mr Quartermaine's year of office was over the members would owe him an even greater debt.

He then requested the new President to take the Chair.

Mr W. K. Wallace, Vice-President, moved the following resolution :—

" That the members present at this meeting desire, on behalf of themselves and others, to record their high appreciation of the services rendered to the Institution by Dr W. H. Glanville during his term of office as President."

He said that he had been a rank-and-file member of the Research Committee under Dr Glanville's chairmanship and had also served as a Member of Council during Dr Glanville's year of office as President ; he was sure that he spoke on behalf of all his fellow-members of Council when he said that Dr Glanville was an exceedingly pleasant man to work under ; he had never taken advantage of his position as Chairman, but had always tried to carry the Council along with him as a team.

Dr Glanville had now vacated the Presidential Chair, and all the members of the Institution would wish to give him their best thanks for all that he had done during his year of office and would hope that he would not find life empty in the years to come.

Mr H. J. B. Harding, seconding the motion, observed that nearly all really nice people were inclined to be diffident, so that members had watched with sympathy and admiration the way in which Dr Glanville had gone about his difficult task. He had set an example in his Presidential Address by making some bold and challenging statements. It was to be hoped that other engineers employed by the State would be encouraged to speak out boldly, without fear or favour, when it was judicious to do so.

Dr Glanville was both an engineering scientist and a scientific engineer, and it was a good thing to have such a man as President in these days, when pure science and engineering showed a tendency to diverge and scientists seemed rather conscious of a superiority.

In his speech at the Annual Dinner, Dr Glanville had remarked that when he first joined the Institution the Council, which used to sit in serried ranks, had an antique and almost apostolic appearance. Everyone hoped that Dr Glanville would be spared as a distinguished Past-President for many years, until he became as apostolic as the great men of his youth.

The motion was carried with acclamation.

Dr W. H. Glanville, in acknowledging the vote of thanks, said that it had been a very happy and a very full year. He had been most fortunate in having a loyal Council, who had given him unstinted support in everything that he had wished to do. He felt that if he had approached in any way the measure of success which Mr Wallace and Mr Harding had indicated, the credit was due to the Council and to the staff of the Institution, and he wished to express his personal thanks to them for all that they had done.

The President then delivered the following Address.

PRESIDENTIAL ADDRESS OF

Allan Stephen Quartermaine, C.B.E., M.C., B.Sc.(Eng.)
President, 1951–52

The progress of an engineer to the office of President of the Institution of Civil Engineers, the unrivalled honour of our profession, can be aided or hindered by the whims of fortune, and in this respect I count myself as specially favoured. As a Student, 45 years ago, I was not directed, as we might be today, to set as a target the distinction of becoming your President. You have, however, been good enough to confer on me this great honour, and I desire at once to express to you my sincere appreciation and deep gratitude, and to assure you that I am very mindful of the responsible task which lies before me. Remembering the eminent engineers who have held this office, I enter upon my duties with pride and humility, and the earnest hope that I may be able to serve the Members, the Institution, and the profession, in a manner worthy of your trust.

Our Past President, Mr Robertson, in his notable Presidential Address in 1949 covered the progress of civil engineering on the railways of Great Britain during the last 20 years, and volunteered to leave sufficient matter for a future Presidential Address by a railway engineer. Knowing his generous nature I sought for the leavings, but met with little success. I was, however, heartened by Robert Stephenson's prophetic words in his Address to this Institution in 1856, when he said :

> " Our British Railways present a fertile theme for observation."

I am accordingly venturing to take the early days of railway construction as a background, before making reference to some of the conditions with which we are now confronted.

As so much of my life has been associated with what was formerly the Great Western Railway, it would, I feel, be appropriate to select as an example of early railway construction some of the works of that famous engineer, I. K. Brunel, a contemporary of that other famous railway engineer, Robert Stephenson, who started the construction of the first main line railway from London to Birmingham just before the inception of the Great Western Railway.

THE PAST

Preliminary Work

In March 1833 Brunel was appointed Engineer for the construction of a railway from London to Bristol, and it is noteworthy that he had refused to allow his name to be considered, when it was proposed to select the Engineer by means of a competition amongst the candidates for the lowest estimates for building the line. His views on the proposal were definite. "You are holding out a premium," he said, "to the man who will make you the most flattering promises. It is quite obvious that the man who has either the least reputation at stake, or who has most to gain by temporary success, and least to lose by the consequences of disappointment, must be the winner in such a race." It is fortunate for us today that attention was paid to these words. Brunel was selected, and we are now reaping the benefit of his foresight and skill, and the hard work and careful thought he gave to the construction and future success of the railway.

Isambard Kingdom Brunel, born in 1806, the only son of Sir Marc Isambard Brunel, had the benefit of a liberal education, followed by engineering training under a father who was an engineer of exceptional ability. From the age of 17 young Brunel was regularly employed in his father's office, and at 20 was Resident Engineer during the later and more difficult period of the construction of the Thames Tunnel. This work involved so many difficulties, emergencies and failures, that it is not surprising that Brunel learnt what was to serve him so well throughout his life—endurance and self-reliance. Although he had this good start in his career, his appointment as Engineer to the Great Western Railway at the early age of 27 was remarkable proof of the Directors' confidence in his strength of character and ability to solve the problems facing the pioneers of railway construction.

Before the Bill for the railway could be deposited full financial support was necessary, and as this was not forthcoming in time, a Bill for a curtailed project was deposited in November 1833. This was for one railway from London to Reading and another from Bristol to Bath. The Bill was thrown out by the House of Lords, but during its passage through Committee in the Commons, Brunel, whose cross-examination lasted 11 days, excelled himself in dealing with the varied and violent opposition. Amongst the opponents were those interested in canals, rivers, and stage coaches; land owners and farmers near London who feared competition with produce from a distance; those who wanted the railway nearer to them, and those who wanted it further away.

By the following autumn sufficient money had been raised to enable the Directors to deposit a Bill for the complete line from London to Bristol, thereby avoiding one criticism of the previous proposal that the railway had a head and a tail, but no body, and was neither "Great" nor "Western," nor even a "Railway." The principal objection now centred on the location, which had been determined with that skill and courage displayed by Brunel in all his works.

His idea was to build a line of very easy gradients and curves, except where this was physically impracticable towards the Bristol end of the line. Here he concentrated his rather steep gradients of 1 in 100 in two comparatively short lengths totalling 4 miles, so that if assistant engines were needed, their use would be restricted to this limited portion of the 118 mile journey.

Apart from these two inclines, the gradients consisted of 71 miles of level or flatter than 1 in 1,000, and the remaining 43 miles not steeper than 1 in 660. Similarly the curves for the first 100 miles from London were very flat, there being only a short length as sharp as 85 chains radius, the rest being between 100 and 500 chains.

The opponents of the Bill contended that the railway to Bristol should be by means of an extension, from Basing to Bath, of the railway sanctioned the previous session by the Southampton Railway Act. Brunel maintained that the gradients of a line from Basing to Bath would be unsatisfactory, but the opponents, whilst admitting that the gradients would be more severe, stated that they were so balanced that the rises and falls compensated one another and the line was therefore in effect practically level. The Chairman of the Committee commented that on this theory the Highlands of Scotland would be as good as anywhere for the construction of a railway.

Another strong line of attack was centred on the proposed tunnel of nearly 2 miles in length at Box, which was described as "monstrous and extraordinary," "most dangerous and impracticable." Though Brunel's persuasive powers overcame this opposition also, it is recorded that when the line was opened there were many passengers who left the train before reaching Box Tunnel, and finished the journey by road.

The greatest contest was in the House of Lords where the battle was fought in Committee for 40 days, the opposing Counsel, Mr Sargent Mereweather delivering a 4-day speech, during which he claimed that almost every conceivable injury would be inflicted by the G.W.R.—the Thames would be choked up for want of traffic, the drainage of the country destroyed, and Windsor Castle left without water. Eton College would be ruined, as London's most abandoned inhabitants would come down by train and pollute the minds of the scholars, and the boys themselves would run up to Town and indulge in all forms of dissipation. The beauty of the country would be destroyed, and the interests of the public better served by a railway from Basing to Bath.

In spite of all these and many other gloomy prophecies the Bill was finally passed, receiving Royal Assent on the 31st August, 1835.

Construction—London to Bristol

Construction of the railway then proceeded at a speed which we cannot but fail to admire when we take into account the scarcity of assistants with engineering experience, and the almost complete absence of mechanical equipment. On the other hand Brunel did not suffer from shortage of labour, nor from the delaying action of Town and Country Planning Acts, and other retarding controls, his principal problem being the design of his works and the method of their execution.

Started late in 1835, the line from London to Maidenhead, 24 miles long, was opened in June 1838; to Twyford, another 7 miles, in July 1839;

to Reading, a further 5 miles, in March 1840; and to Chippenham, a total of 94 miles, in May 1841. Concurrently the line from Bristol to Bath was opened in August 1840, and the final line from Bath to Chippenham, including Box Tunnel, in June 1841—a total length of 118 miles in under 6 years.

May I now refer briefly to just a few of the works involved, taking them in order from London, although construction proceeded simultaneously from both ends of the line.

The first contract at the London end was for the viaduct across the Brent Valley near Hanwell, let in November 1835, 2¼ months after Royal Assent to the Bill, and completed in 18 months. It is 300 yards long and 65 feet high, comprising eight elliptical arches of 70-foot span, each pier being formed of twin pillars slightly tapered on all faces, and surmounted by stone capitals. The photograph shows the handsome appearance of the work, known as the "Wharncliffe Viaduct" after Lord Wharncliffe, who was so helpful when the Bill for the railway was before the House of Lords Committee.

The next structure worthy of special mention is the brick arch bridge over the River Thames at Maidenhead, a standing tribute to the skill and courage of Brunel, and to his pleasing sense of proportion. The bridge is formed of two flat elliptical arches each of 128-foot span, and a rise of 24 feet with four semi-circular arches of about 28-foot span at each end. The work was started in 1837, and the centering eased after 12 months, which operation delighted the critics as the eastern main arch distorted slightly, though the western one was unaffected. The contractor admitted his error in easing the centering too soon, and rebuilt the defective portion of the eastern arch. The centering was then left in position until late in 1839 when it was blown down by a violent storm, leaving the critics once more in despair as the arch and the whole of the bridge remained in perfect condition. Since then it has required only superficial maintenance, and carries today the heaviest modern locomotives and high speed traffic.

When it became necessary in 1890 to quadruple the line, the widening was carried out to the same design by an addition on each side, the stone string courses and capping being refixed or reproduced on the new up and down stream faces. Tie-rods 1¼ inch in diameter were inserted at regular intervals across the bridge between the new spandrels, but not so as to show on either face.

Nine miles further on between Twyford and Reading is the start of the cutting through Sonning Hill, nearly 2 miles long, with an average depth, for half a mile, of nearly 60 feet, involving a total excavation of about 1¾ million cubic yards. The first contractor started at the end of 1837, but failed to make the promised progress and was replaced by three others, one of whom also failed early due to trouble with his men. The work was ultimately completed in December 1839 by direct labour under a Resident Engineer, and included two bridges across the deepest part of the cutting, one a 3-arch bridge for a turnpike road, and the other a typical Brunel design timber bridge for a minor road.

From this point westwards to Box Tunnel, 100 miles from London, the works were considerable and varied, including apart from stations, lengthy and high embankments, and deep cuttings, many of which gave trouble due to the unstable nature of the clay; a variety of bridges, including two over the Thames at Basildon and Moulsford, each consisting of four 62-foot brick arch spans; a stone viaduct just west of Chippenham, followed by a high embankment for over 2 miles, and then 3 miles of almost continuous deep cutting leading to the tunnel under Box Hill.

This tunnel, the subject of so much criticism during the passage of the Bill, was the longest contemplated at that time. It was a measure of Brunel's confidence in his own judgement and ability to surmount difficulties that his location required the successful completion of this formidable work and the subsequent operation of steam locomotives through the tunnel, nearly 2 miles long on a gradient of 1 in 100.

The strata to be penetrated consisted of great oolite or Bath stone at the eastern end, followed westwards by light clay, inferior oolite and marl. The half mile length through the Bath stone forming contract no. 1 was to be left unlined, the remainder, contract no. 2 being lined with brickwork throughout. The six permanent and two temporary shafts, varying in depth from 70 to 260 feet, were completed in time for the two contracts to be let for the tunnel construction in 1838.

Apart from two steam pumps, which were used to deal with the heavy influx of water, men with horses, manual tools, and gunpowder, accomplished the work. The excavated material was drawn up the shafts by horses turning rope drums, and at the peak of the work four thousand men and three hundred horses were employed. The flow of water from numerous fissures impeded progress, and for 8 months a section of the work was stopped as the water gained on the steam pump, flooding the workings and rising to a height of 56 feet in one of the shafts. This was eventually overcome by the installation of a second steam pump of 50 horse-power. In February 1840, when two-thirds of the tunnel was constructed, three additional shafts were sunk to provide more working faces, as completion was becoming increasingly urgent. Brunel and his Resident Engineer, William Glennie, forced the pace, and by continuous night and day work the tunnel was finished ready for the opening of the last section of the line in June 1841.

The Inspector-General of Railways, Sir Frederick Smith, reporting on the line on the 28th June, 1841, described the unlined portion, which in places is 40 feet high, as "formed by excavating the natural rock in the shape of a gothic arch, no part of which is lined with masonry." Much fear was entertained in regard to the safety of the tunnel, and this was increased by the statements of an eminent geologist, Dr. Buckland, at a meeting in this Institution in May 1842, when he expressed the view that those portions of the tunnel constructed through the great oolite and left unsupported were peculiarly liable to danger, and he feared that the vibration caused by railway carriages would tend to loosen and detach masses of stone. This resulted in a special examination being made by Sir Frederick Smith's successor, Major-General Pasley, who upheld Brunel's opinion, stating that he was perfectly satisfied with the security of the tunnel. After a severe frost in 1845 a small piece of rock fell from the bottom of one of the temporary shafts, and arches were turned under this and one other shaft. About 50 years later a further section was lined after a fall following an exceptional frost, thereby reducing the unlined portion to about half its original length.

As will be seen from the drawings of the west portal, the persistent criticism, evil forebodings, and the many difficulties of construction did not deter Brunel from providing an attractive entry to this "monstrous and most dangerous hole."

The works between the tunnel and Bristol were generally more varied and heavy than those on the London side of Box. They include a second tunnel near Box, 200 yards long, a handsome arch bridge of 90-foot span over the River Avon at Bathford, a long and high retaining wall holding up the Kennet and Avon Canal, two short tunnels and a viaduct of thirty-three arches leading to an 88-foot span arch bridge over the Avon at Bath Station, which itself was built on arches.

Leaving Bath the railway again crossed the Avon by two 80-foot span timber arches on a 60-degree skew, followed by a viaduct of seventy-one arches, another part viaduct and part retaining wall ½ of a mile long, and a walled cutting leading to Twerton Tunnel, 264 yards long. After this there were 2 miles of high embankment, a deep cutting, Saltford Tunnel, 176 yards long, another long embankment and deep cutting, two short tunnels subsequently opened out, then a 1,000-yard tunnel at Brislington and two shorter tunnels, one of which was also opened out at a later date. Finally in the last mile there were bridges over the Avon, the Feeder Canal, and the Floating Harbour leading to the terminus at Bristol, which like Bath was built on arches. The original station at Bristol forms today the present nos. 13 and 15 platforms, and the roof, still in good condition, is an example of handsome timber work spanning 72 feet between columns with an overall width of 112 feet between the outer walls.

At Paddington the original station was built in 1838 in rural surroundings just west of Bishops Road, on the site which Brunel intended for the Goods Depot. This was due to the delay experienced in obtaining powers for the road diversions necessary to enable the station to be built on its present site. In 1850 the Directors, influenced by the increasing traffic, decided to provide a more suitable terminus, together with administrative offices and Hotel. Brunel was greatly attracted by the engineering possibilities of this, as evidenced by the following extract from a letter he wrote to a friend on the 13th January, 1851 :

> "I am going to design in a great hurry, and I believe to build, a station after my own fancy—that is with engineering roofs, etc., without any architectural attempt. It is at Paddington in a cutting and admitting of no exterior, all interior and all roofed in."

The station was opened in 1854 and is, I suggest, an admirable example of Brunel's ability to design engineering structures of simple and graceful appearance.

The original roof which forms the major portion of the present station was 700 feet long by 240 feet wide, comprising a centre arch span of 102 feet rising to a height of 62 feet, and two side spans of about 69 feet each, 43 feet high, crossed at two points by transepts 50 feet wide. In addition there was on both the up and down sides a further 50-foot width of flat roofing 20 feet high.

The three arch spans are roughly elliptical in shape and composed of wrought iron ribs at 10-foot centres springing from light lattice girders supported on columns 30 feet apart. The ribs taper from about 18 inches deep at the crown to 2 feet 6 inches near the springing, and are formed of light angles and web plates with ornamental holes cut in the webs round the soffit to enhance the light appearance. The wide expanse of glazing running for the full length of all spans and transepts, and occupying nearly 70 per cent. of the roof area, was of the ridge and furrow style adopted by Sir Joseph Paxton in his 1851 Exhibition building. The general effect was, and still is, bright and pleasing. The roof covering served the requirements of the station until 1914 when an additional span of 109 feet of the same design was added on the arrival side.

Since the roof was completed in 1854 no serious maintenance work, other than painting and resheeting, was needed until 1914 when some of the cast iron columns were replaced by built-up steel ones, the remainder being replaced in 1924. Since then slight repairs have been carried out to some of the ribs, and the glazing renewed in two spans. In 1940 and 1941 tie rods were provided at arch springing level to minimise the effect of bomb damage, and in 1944 a rib in the eastern transept was severed by a bomb, without consequential effect on the remainder of the roof, one 20-foot length of one rib only having to be replaced. The roof in other respects remains today substantially as constructed by Brunel nearly 100 years ago.

There is not time this evening to refer in any detail to the many other works of Brunel, including his notable timber viaducts in Cornwall, his unsuccessful atmospheric system of propulsion for the South Devon Railway, his first steamship the "Great Western," 2,300 tons displacement, launched in 1837, and his famous steamship the "Great Eastern," 680 feet long and 27,400 tons displacement, launched broadside in 1858. I cannot, however, omit reference to the last and greatest of his railway works, the Royal Albert Bridge across the River Tamar at Saltash, extending the Great Western Railway from Devon into Cornwall.

Royal Albert Bridge

A proposal was made in 1844 to take trains into Cornwall by means of a train ferry, but in 1847 Brunel stated it would be practicable to bridge the Tamar at Saltash where the river is about 1,000 feet wide and 70 feet deep at the centre at high water. After consideration of the site conditions and the requirements of the Admiralty for naval vessels passing up and down the river, Brunel finally decided on two main spans of 455 feet with a centre pier in deep water, and seventeen approach spans making a total length of 2,200 feet, the main spans being 100 feet clear above high water.

The bridge was built to carry only one track, Brunel's view being that in the case of such an expensive structure the extra cost of a double line, £100,000 at that date, could not be justified for this short length. It remains today the only piece of single line between Paddington and Penzance, and is not really an inconvenience as the traffic margins are sufficient.

Apart from the design and erection of the main spans there was the difficult problems of constructing the centre pier. As a description of this work is given by R. P. Brereton, Brunel's principal assistant, in his Paper to the Institution in 1861, I will only say that it included the sinking of a wrought iron cylinder 37 feet in diameter, 90 feet high, and weighing 290 tons, in 70 feet of water through about 16 feet of silt to reach a reliable foundation on the rock in the middle of the River Tamar. Within the cylinder was constructed the masonry pier 35 feet in diameter, carried up to 12 feet above high water level. On this stand four cast iron octagonal columns nearly 90 feet high supporting the upper arched section of the pier, through which the trains pass and on top of which the tubes are carried.

The two main spans each 455 feet long were designed as a development of an earlier Brunel bridge over the River Wye at Chepstow, in a way which made use of a wrought iron tubular arch in effective combination with suspension chains, so that the pull of the chains is resisted by the tubular arch. The track-carrying girders are suspended on hangers from the tubes and the chains, alternate hangers being connected to both tube and chain. The hangers are braced transversely and longitudinally and the finished truss forms a remarkably rigid structure, but presents us with an extremely difficult problem in stress calculation.

In the general design Brunel made the rise of the tube, 28 feet 9 inches, approximately the same as the fall of the chains, 27 feet 6 inches, both forming flat parabolic curves and giving a pleasing appearance. The arch is a single wrought iron oval tube 16 feet 9 inches wide and 12 feet 3 inches high, from which hang four suspension chains each having a cross section of fourteen 7-inch × 1-inch wrought iron links. The weight of the ironwork in each span is over 1,000 tons.

The erection was carried out by constructing the trusses complete on the Devon shore, floating each one out in turn on pontoons to its correct position where it was jacked up in short lifts, the masonry of the shore pier and the cast iron columns of the centre pier being built up under the truss as the work progressed. The total lift was nearly 90 feet, and as the trusses are 455 feet long, 72 feet high, and less than 20 feet wide, the work must have been planned and executed with great care, particularly as the erection occupied 12 months, and the site is open to severe winds. With the trusses in their final position the top of the tubes is 260 feet above the river foundation which is 35 feet in diameter. Brunel was present during the early stages of the lifting of the first truss, and thereafter the work was supervised by his distinguished assistant, R. P. Brereton.

The bridge cost £225,000 and was opened by the Prince Consort in 1859. Since then the decking of the bridge has been renewed, the approach spans reconstructed, additional horizontal bracing added to the main trusses, and some repairs carried out at the attachment of the verticals to the tubes, otherwise the bridge remains as constructed, a monument to the skill and courage of the designer.

After this necessarily brief review of some of Brunel's works I would like to bridge the gap of nearly 100 years to 1951 by referring to three characteristics of this famous engineer, which are perhaps of particular interest today—his energy, his artistic inspiration, and his wide general knowledge.

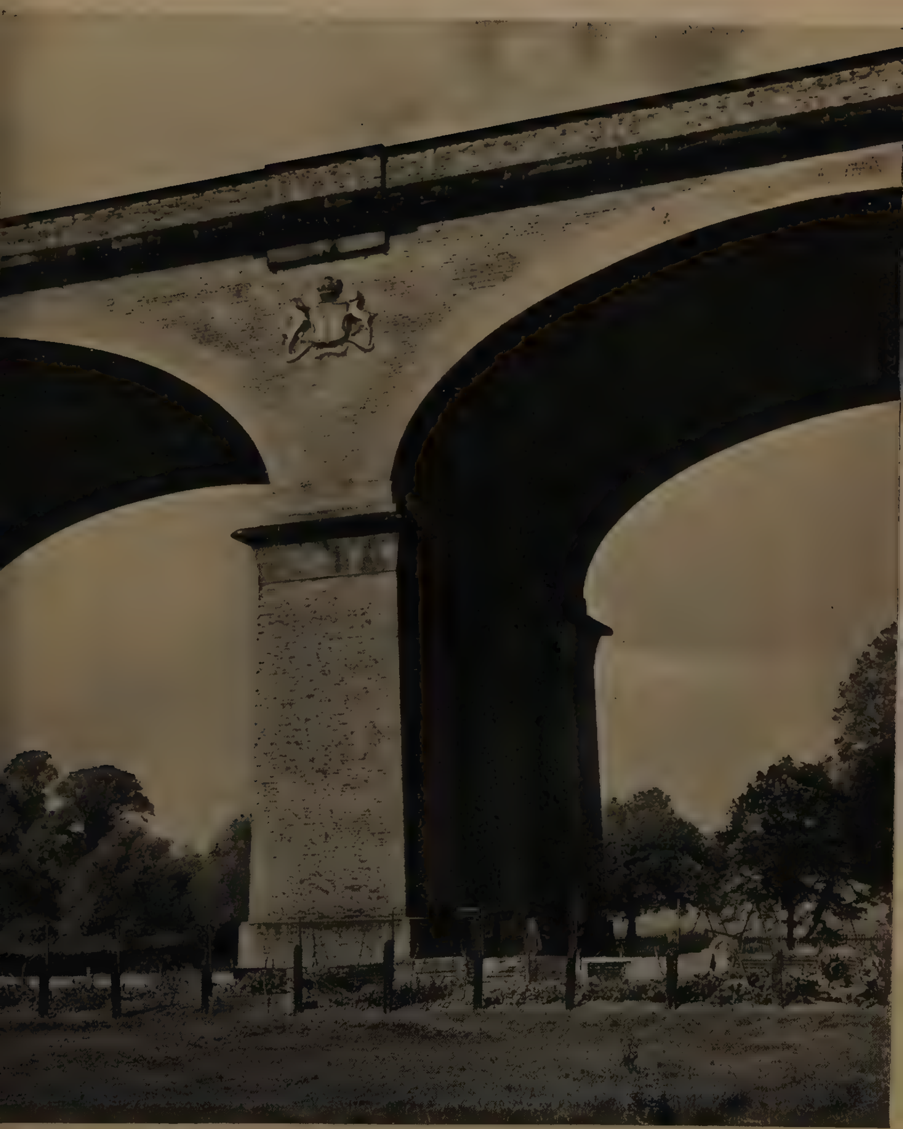
Brunel, who was a senior Vice-President of this Institution, and would have become President but for his early death at the age of 53, displayed throughout his life almost tireless energy and enthusiasm for his work, and much of the comfort and leisure we now enjoy is derived from the strenuous labours of men of his tenacity. The spread of industrialism to almost all countries, the effect of two world wars, and the present tragic international situation is depriving us of our advantages. The position today demands that we call to mind the indomitable character of those who helped to build our industrial prosperity, as it seems evident that we can only hope to retain the benefit of our heritage by a revival of the unconquerable spirit of the Battle of Britain and the energy of such men as Brunel.

Brunel's works stand as proof of his designing skill, and also show how much thought he gave to the appearance of his structures, using to the full his remarkable eye for proportion. This has not always been followed by his successors, and as Members will know, six valuable lectures were delivered at the Institution in 1944 on the "Aesthetic Aspect of Civil Engineering Design," with the object of stimulating within civil engineers and students an appreciation of this phase of their work. The key-note today is economy and we find it beyond our means, even if it were our desire, to provide the fine stone capitals seen in such structures as the Wharncliffe Viaduct, but the beauty of much of Brunel's elliptical arch work was achieved without additional cost, and without the assistance of the Royal Fine Art Commission, or the approval of the Town and Country Planning Authority.

Though Brunel had very many difficult technical problems to solve he did not restrict himself to one at a time, as for example when he built the steamer "Great Western" at the very time when he was engaged on the construction of the Great Western Railway. Nevertheless he found time to study nature and art, and to acquire knowledge of the world and its affairs, and combined administrative ability with his exceptional engineering talent. Today the field of engineering knowledge and research is so wide that it encompasses not only the whole of an engineer's working time but much of his leisure hours to the exclusion of other interests, and it would indeed be wrong if too many of our Members deprived the profession of their skill by undertaking other activities. On the other hand, the increasing extent to which the work of the engineer affects almost every aspect of the life of the community requires that he should be more frequently amongst those who hold administrative posts, and are represented on the governing bodies of the nation. The Engineer's readiness to get on with his own job, and to leave to others the less arduous task of talking about it, is understandable, but this natural inclination, increased by the desire to avoid anything approaching self-advertisement, results in his being too readily and too automatically excluded from the administrative side of affairs. We do not want a Board of Directors, a County Council, or a House of Commons composed of engineers, as this would be not only a scandalous waste of engineering talent, but as unsatisfactory as a House full of doctors, accountants, scientists, or even professors of economics. On the other hand the present scanty representation of professional engineers in administration and government is neither good for the country nor the profession. It lacks originality to say that the time is overdue for this to be changed, but the need grows as the engineer's work becomes more widespread.

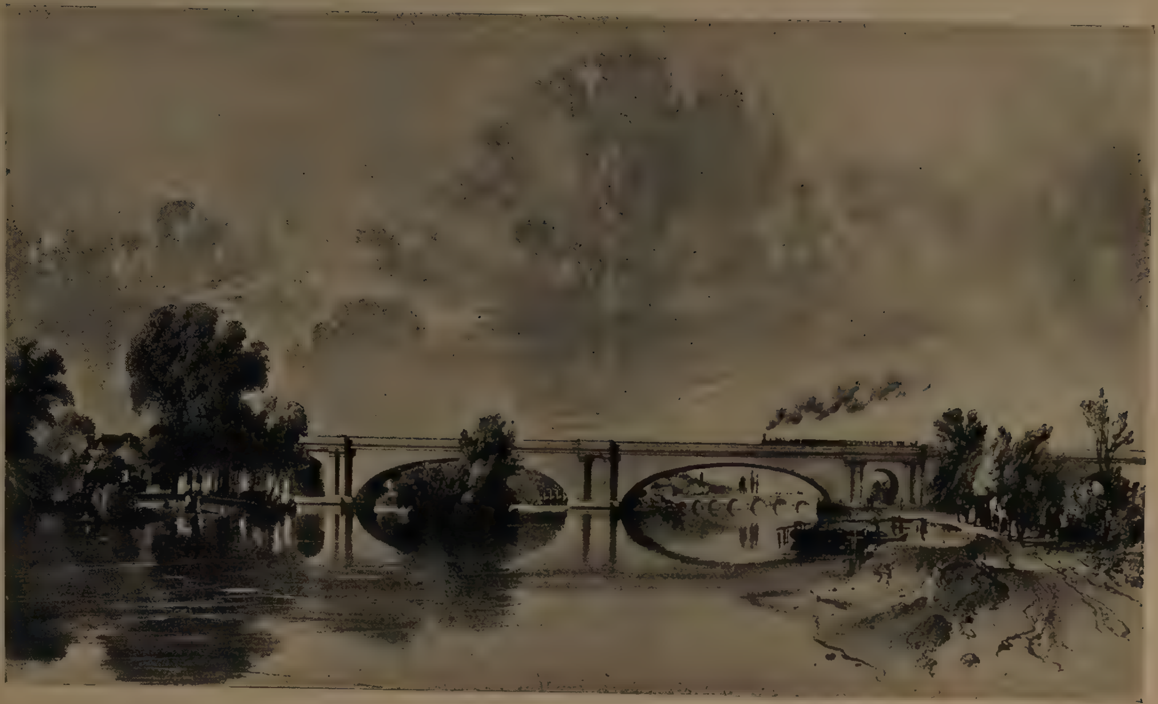
The breadth of view and knowledge needed for successful administrative work is, however, not acquired by technical education alone. Contact with those engaged in a variety of activities, and interest in social science, philosophy, law, economics, industrial relations, and even the art of speaking and report writing are all helpful, and to some extent necessary. The Railway Executive have included engineers amongst those who attend Sessions at the Administrative Staff College at Henley, and this is a valuable step in the right direction. In many industries such as the railways, where engineers control a large number of men and collaborate with many departments, administrative ability is required to perform successfully the engineer's own work, quite apart from its value in a purely administrative post. It has to be admitted the technical and scientific information of which the young Engineer has to obtain a working knowledge, is now so varied and extensive, that one hesitates to suggest he should add to this a determination to acquire more general knowledge. It may sound un-

Fig. 1



harncliffe Viaduct showing Wharncliffe Coat of Arms and Widening on Far Side

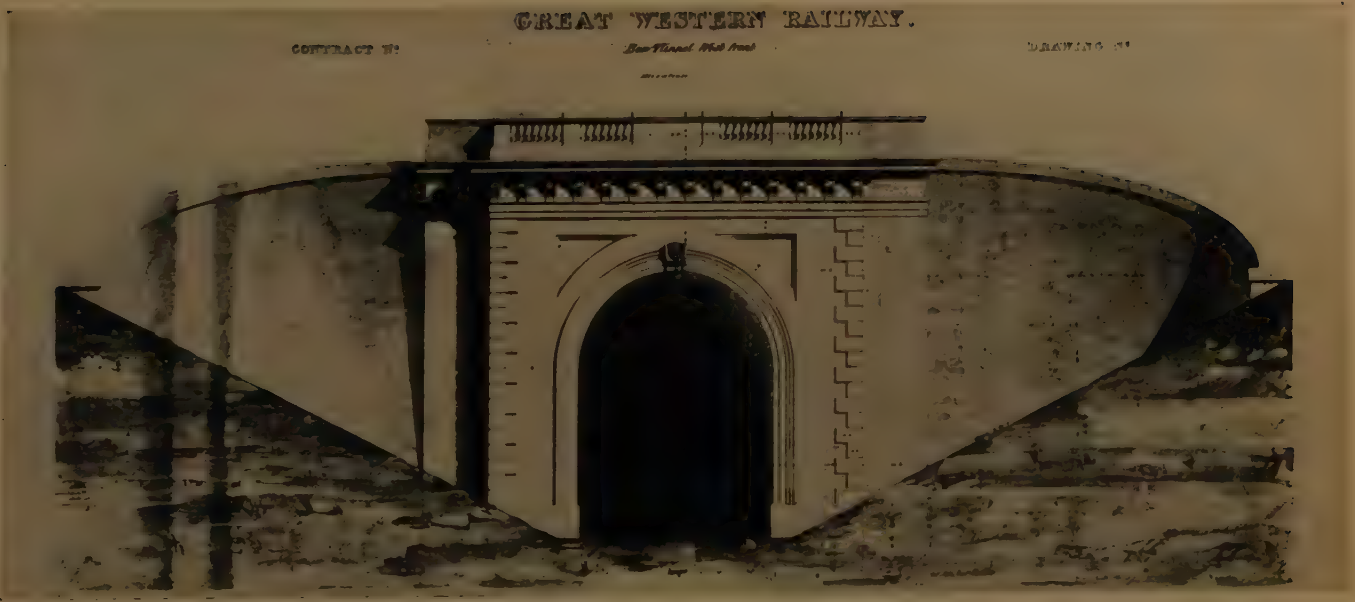
Fig. 2

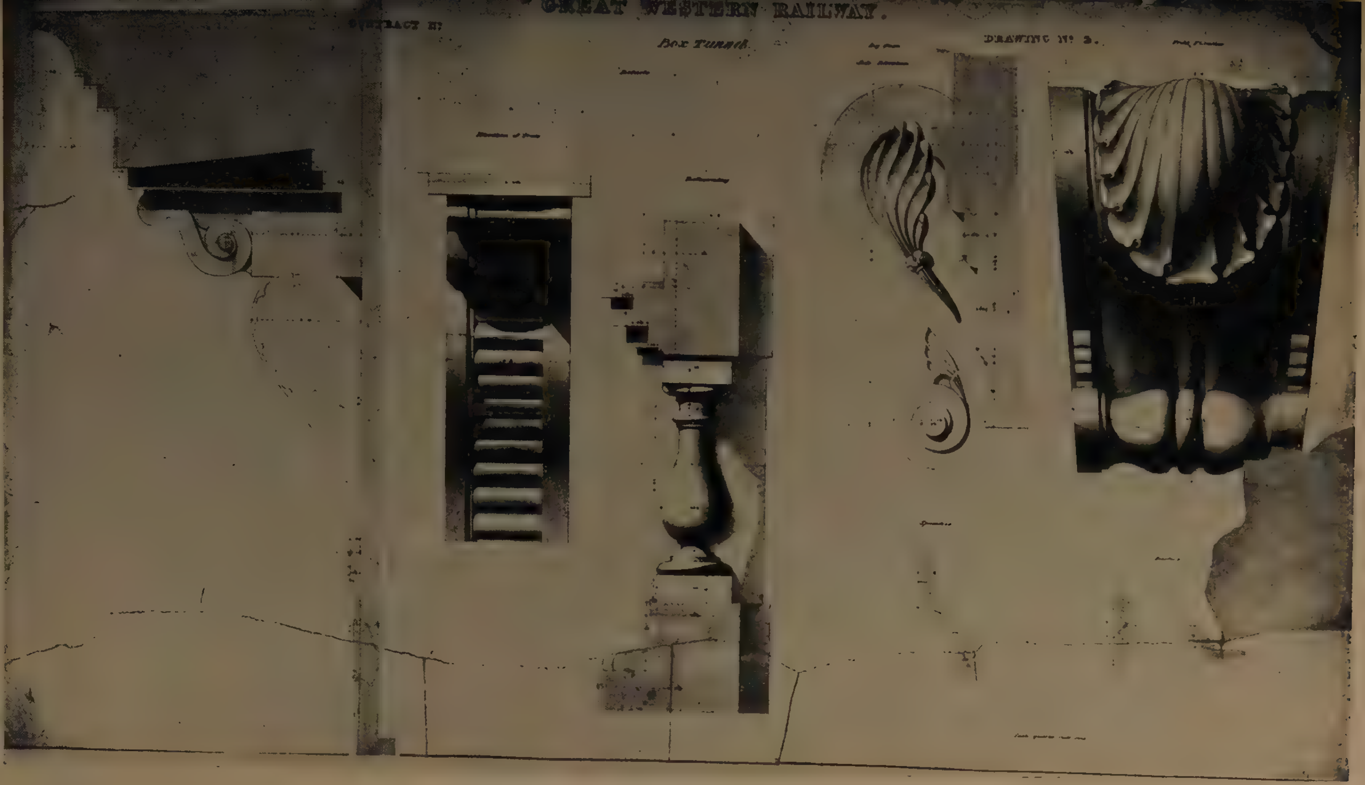


Reproduced from J. C. Bourne's "History and Description of the Great Western Railway"

THE MAIDENHEAD BRIDGE

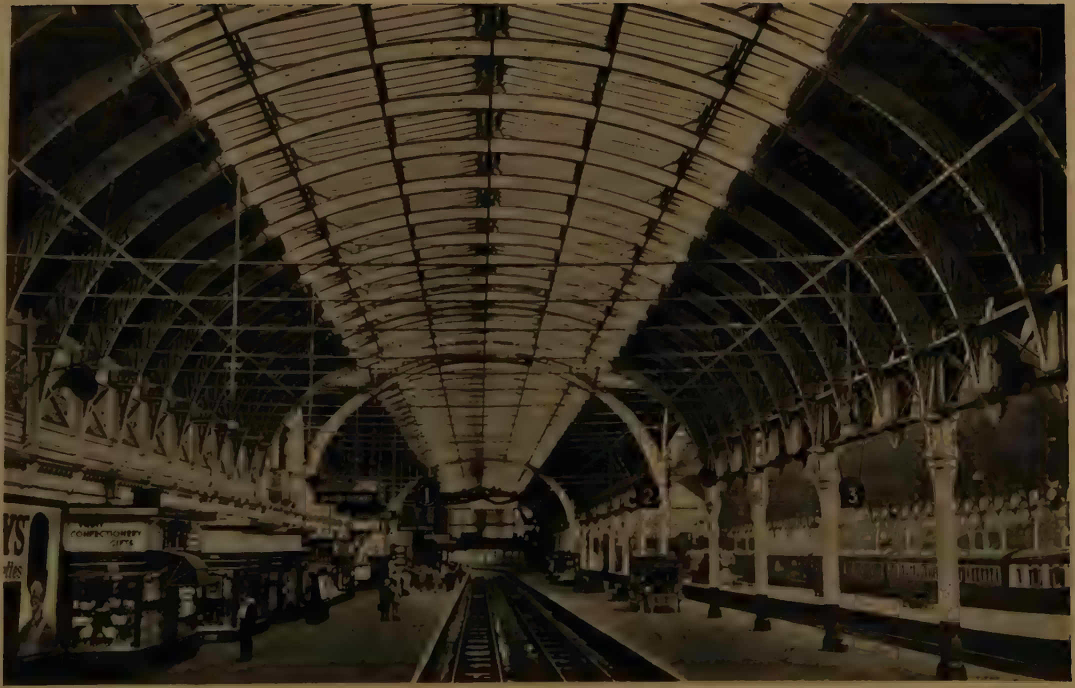
Figs 3


GREAT WESTERN RAILWAY.
CONTRACT No
Box Tunnel West front
Elevation
DRAWING No



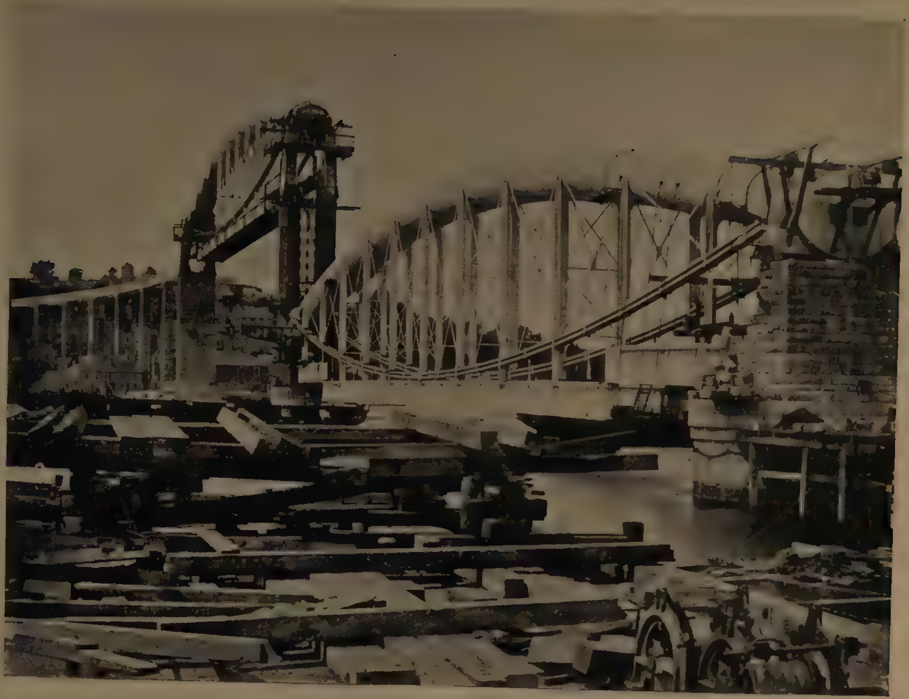
Drawings of the West Portal, Box Tunnel

Fig. 4



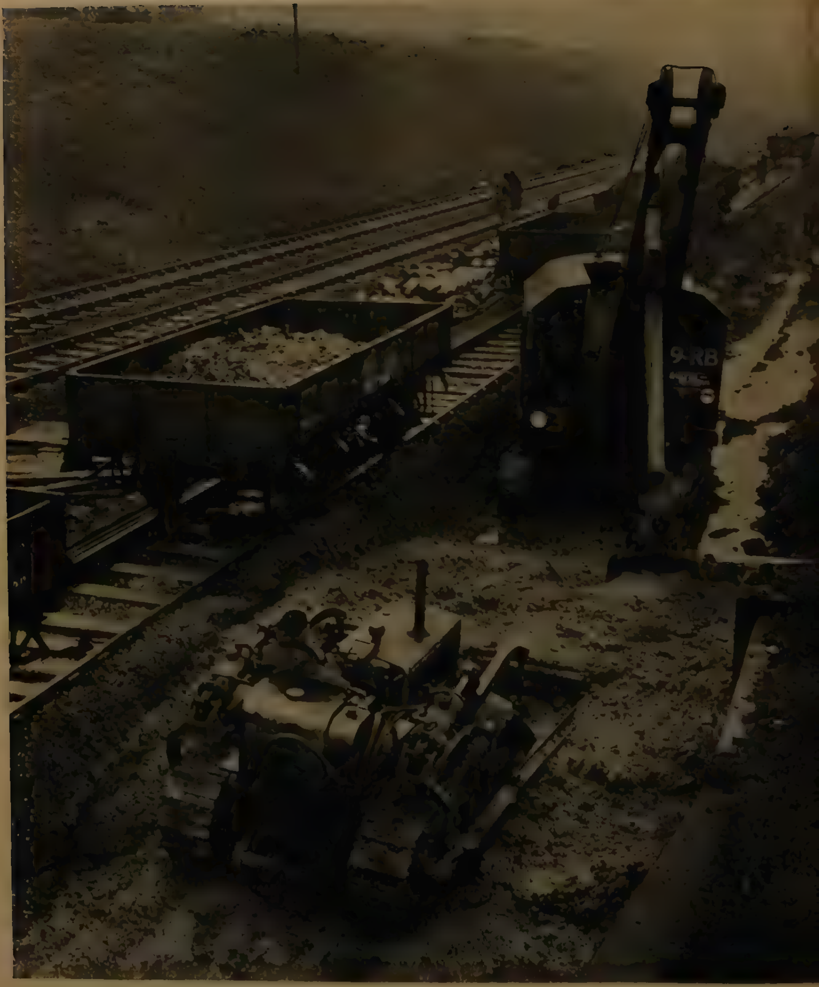
PADDINGTON STATION. No. 1 SPAN

Fig. 5



The Royal Albert Bridge, Saltash. Erection 1858

Fig. 6



" OPERATION PORRIDGE "

reasonable, and much cannot be expected in the early stages of his academic career, but it is for the very reason that his technical education never ceases that it is necessary for him to resist the tendency to exclude everything else.

THE RAILWAYS TODAY

May I now turn to the railway situation today.

Railways Essential

During the nineteenth century construction was the great objective and railways were projected and built in every direction, until by the end of the century the country was traversed by a network of lines serving all districts. Now the position is that the railways exist, and it is sometimes said without thought that their days are numbered, as they will inevitably be superseded by road and air transport. You will not be surprised that I do not subscribe to this view, and I would like to give some indication of the position as I see it.

In Dr Glanville's brilliant Presidential Address he referred to the congestion which already occurs on the roads, particularly in and around large cities. A 5-year programme has since been recommended to the Minister of Transport by the London and Home Counties Traffic Advisory Committee, involving an initial expenditure of about £20 million in London at the earliest possible moment, and confirming the approaching critical situation in the London area. Every addition to road traffic adds still more to the inevitable difficulties which must arise in a country the size of Great Britain when the number of vehicles on the roads is increased at a much greater rate than the improvement of roads to carry them. Partly as a result of two world wars the rate of production of road vehicles has been increased, and the construction of roads had been delayed through lack of labour. It may be said that this will cure itself in a few years by expenditure on road improvements throughout the country, and much could no doubt be done in this way. What is really happening is that construction work, even on arterial and by-pass roads, which was too slow before the war, is now by force of circumstance reduced beyond recognition. The more serious problem, however, of dealing with any considerable increase of road traffic into major towns is a far greater one involving unpredictable delay and vast expense.

In the London area surface traffic at peak hours has reached saturation point and for such traffic further tube railways will doubtless prove the quickest and cheapest, and probably the most certain means of improvement. It seems essential that an early start should be made with the most urgent section of new tube railway, followed by steady progress with a long term programme.

Most of us are aware that the main line railways carry an immense amount of traffic, though it may not be remembered except at holiday times. In 1950 the main lines handled 1,000 million passenger journeys and 22,000 million net ton-miles of freight. During the morning rush hour at the main line termini in London 3,300 passengers arrive per minute. If this traffic were gradually transferred to the roads it is clear that transport thoughout the country and the approach to large towns would become chaotic, in spite of all the prodigious efforts which might be made, and the crippling expenditure which would be incurred in an attempt to match the roads to the traffic. Lengthy journeys by car at busy times already involve joining queues travelling behind slow moving lorries, whilst each car awaits an opportunity to pass in safety. On the long stretches of main roads of two car width the delay in passing is often prolonged, and a further queue is reached at no great distance ahead. If the railway passengers and freight which I have just mentioned were added to these queues and to the congestion entering and leaving large towns, the temperatures of cars and drivers would rise, the speed of transport would fall, and the disastrous story of road accidents, estimated to cost about £100 million per annum, would become an even greater national tragedy. Furthermore, the strategic value of the railways would be lost, and the burden carried by them during the war, when the traffic was so heavy that intending passengers were exhorted to avoid rail travel, would, if placed upon the roads, be a menace to our security.

In mentioning these extreme conditions it is not my intention to argue a case for rail in preference to road traffic, but to indicate that both forms of transport are essential in this highly industrialized and heavily populated country. The British Transport Commission are applying themselves to the difficult task of integrating all forms of transport, and this is clearly the urgent problem with which we are faced.

Capital Investment

The existing railway system, which would cost about £3,000 million to construct today, is an invaluable heritage which must be not only maintained and used to the best advantage, but also improved wherever necessary to ensure that the country reaps the greatest benefit from it. By this I do not infer that there are not many railway branch lines and stations which can no longer justify their existence. Such deadwood should be, and is being, cut out of the railway tree. The rest of the tree, however, should be nurtured and fertilised, and not starved into a state of unfruitfulness to the everlasting detriment of the nation's economy. Reduction in operating and maintenance expenditure is essential, but wise capital investment on improvements is equally necessary. The confidence of all associated with the actual running of the railways needs to be strengthened by the certain knowledge that the industry must and will advance, and that policy will be directed to this end. The present unavoidable curtailment of capital expenditure, if pushed too far, may not only retard the postwar convalescence of the railways, but set them back so far as to cripple them permanently. With all the commitments facing the country, efficient transport by rail and road is of first importance and expenditure on revitalizing this in a co-ordinated manner is imperative. The late Minister of Fuel and Power said that rearmament must not be allowed to check investment in the mines because coal is a vital defence requirement. May I add " so also is transport." The Railway Engineer desires to assist in the wise planning and speedy execution of construction works to provide better, quicker, and if possible, cheaper service. By various methods he is achieving some success in handling the problem of labour shortage, but like many others he wants a far higher place in the queue for materials.

Maintenance

The ever present problem, to which the engineer directs his thoughts and energies, is the vast and varied task of maintaining the existing way and structures. In general this might be described as the more economical design, use of materials, and execution of civil engineering work, the better organization of the considerable manpower employed, constant research into improved methods, and the most effective application of mechanical equipment. The annual civil engineering bill for maintenance is over £50 million and to ensure every effective reduction in this without loss of efficiency necessitates the employment of staff with engineering training, experience, and ability of a high order.

During the 100 years or more that many of our railways have been in existence, much has been done to improve the standard of construction by replacing original timber and cast iron structures by stronger and more lasting ones, providing heavier permanent way and better curves, improving signalling, track formation, and many other features. This work does not cease with the passage of time, as structures continue to wear out or deteriorate to a point beyond which it is no longer practicable nor economical to repair them, and renewal becomes necessary. In addition many stations, goods sheds, locomotive depots, and similar works need reconstruction on modern lines due to obsolescence. Reconstruction and modernization is normally undertaken in a way which will give the best and most lasting results for the least expense in first cost and subsequent maintenance. On the other hand there are sometimes cases where it is reasonably certain that change of traffic or policy will render a structure redundant in a comparatively short time. Long life is not then one of the essentials and the cheapest building to meet the requirements is all that is needed. It is, nevertheless, necessary to inject some engineering economics into the department asking for cheap buildings, otherwise the money saved in the cheapening process may have to be spent subsequently by the unfortunate engineer on the costly maintenance of these utility models.

The civil engineer has to direct his attention to a great variety of maintenance works, but I will refer briefly to only three—bridges, tunnels, and the formation under the track.

Bridges

British Railways have over 60,000 bridges of all shapes and sizes up to the largest, the Forth Bridge, and of ages from 1 to over 100 years. Many of the old masonry and brick arch bridges and viaducts are almost as good as when constructed; others develop a variety of defects. The more usual, such as bulging spandrels, cracked arches, perished brickwork, and decomposition of mortar, can be dealt with by well established practices. Foundation settlement and distorted arches afford more scope for ingenuity in repair.

Most of the oldest metal bridges are of wrought iron and many are over 90 years old. These have some of the characteristics of the old soldier, but they do ultimately expire, as do the less aged steel structures. There are, in consequence, plenty of opportunities for evolving improved designs based on experience of the behaviour of old bridges, and our present knowledge of riveted or welded steel work, reinforced concrete, and pre-stressed concrete of various forms and methods of fabrication. The present need for conserving steel is an added incentive to the greater use of reinforced concrete, whether pre-stressed or not, and always before us is the desire to find new and better materials and forms of construction, the guiding principle being reasonable first cost, long life, and low maintenance expense. The problem of speedy erection, however, generally affects the design, as the actual replacement of most bridges carrying the railway must be undertaken on Sundays and may have to be completed in a few hours, because trains, unlike lorries and cars, cannot be diverted to side roads for several weeks.

Tunnels

There are on the British Railways 1,085 tunnels with a total length of 310 miles. Many of these are approaching or over 100 years old, and most have withstood the passage of time well. Their principal defects, the deterioration of mortar and scaling of brick or masonry linings, are repaired by deep pointing, or cutting out and replacing one or more defective rings in short lengths as necessary. Less frequently a tunnel may require drastic treatment due to serious distortion of the arch or side walls, involving a new invert or complete relining, or it may even be necessary to construct an entirely new tunnel.

At the present time there is in hand the construction of a new double line tunnel, just over 3 miles in length, on the Manchester–Sheffield line through the Pennines between Woodhead and Dunford Bridge stations. The lining of the existing twin tunnels, after 100 years life, has deteriorated so much that further economic maintenance under intensive traffic conditions has become impracticable. The construction of the tunnel was commenced in March 1949, and the pilot heading completed last May.

Such works are fortunately exceptional, but the railways are never without tunnel problems somewhere. In two recent cases the side walls settled and moved inwards, involving in one tunnel very heavy inverting and relining, and in the other the complete opening out of the length affected.

Track Formation

Efficient and economical maintenance of the permanent way presents a serious drainage problem which is still the subject of experimental research. Water is very frequently a source of trouble to the engineer, and it is particularly so under the railway track. In theory, rain water should drain through the ballast and pass harmlessly to the side drains or ditches. Much of it behaves in this way, but where the formation is a cohesive material such as clay, of low shear strength when the moisture content is high, it is a constant fight between the clays readiness to increase its moisture content beyond stability, and engineers' efforts to remove the water before this occurs. Once the clay gains a lead on the engineer it rapidly takes charge, as the resulting slurry holds the water, and passes into the open joints of drains, sealing these and impeding or stopping further water reaching the drains. This is the beginning of what might be described as " Operation Porridge," as the slurry rises up through the ballast, rendering the engineer almost powerless until he removes the track and digs out the offending mixture of ballast and slurry down to a level at which the clay is in its undisturbed state. On this new formation he must start afresh, carrying out a process known as " blanketing." In the past, a layer of ashes was tried with little success, and fine sand proved to be a better material. Now it is considered that more lasting results will be obtained by a thickness of 1 foot or more of stone dust, rolled or tamped mechanically before adding a layer of coarser material graded upwards to the size of ballast, on which the ballast itself is spread and the track replaced. At the same time the side drains must be renewed at a suitable level to draw off the water before it ever reaches the clay again. Other methods have been tried, including cement grouting, sand piling, sand infiltration, and even laying concrete slabs on the formation. Considerable lengths now require treatment due to neglect during the war and subsequent shortage of labour. Whilst lasting results are essential, we need to find means of expediting the work and reducing the cost. In any event the restoration of defective track formation and drainage cannot be deferred if we are to provide better running and a subsequent decrease in maintenance expense.

The Railway Civil Engineer Today

Improvement of the railways and reduction in the cost of maintaining the way and works can only be achieved by combining hard-headed railway experience with a progressive spirit of research and development. This involves investigation of the problems by skilled and well trained engineers. Mediocrity and accountancy will not be sufficient.

Opportunites are accordingly provided to graduates and young men of matriculation standard for training under agreement in accordance with the Institution requirements; at a later stage selected young engineers showing more than average competence are picked for further special training or for brief tours on Continental railways, or to attend courses or international conferences on engineering subjects.

With the clear necessity for making the railways more efficient, whether it be by the extension of electrification, the mechanization of goods working, the improvement of design to reduce cost, the wider application of modern techniques, the profitable expansion of the use of mechanical plant, or any other advance on older methods, an essential need is for engineers who are skilled in their work and carry it out in a way which inspires their staff with confidence, enthusiasm, and pride of achievement.

It is a compliment to the railways that many young men aspiring to become qualified engineers recognize that railway work is a valuable training ground, and it is necessary that the best of these should not be attracted away from the service. In an organization so vast, care is needed to ensure that centralized instructions and standards, necessary and desirable as they are, do not discourage the inception and development of new ideas amongst the staff of the Regions. Good engineers live and thrive on the success of their ingenuity and inventiveness, and the morale of the many excellent men, young and old, on the engineering staff would be lowered if their initiative were unduly restricted. Any impression that the work of a railway engineer is lessening in interest and opportunities is wrong, and there must be no room for doubt that railway service is attractive as a permanent career.

Technical and organizing ability is more than ever necessary when receipts are not keeping abreast of rising costs. The railways, unlike other nationalized industries, cannot freely raise the price of the commodity they sell, because of the statutory restrictions placed upon them, and because the commodity is not like coal, gas, and electricity, which the public must buy whatever the price. Railway engineers of today are impressed with the importance of reducing costs, and I commend their efforts to achieve this whilst faithfully carrying their heavy responsibilities. No praise or criticism, however, and no amount of skill and training will be sufficient unless we add also something of the spirit of Brunel. He lived at a time when this nation was creating its greatness, and recognized that only by unflinching determination and incessant effort could obstacles be overcome. Such a spirit is needed more than ever today, both on the railways and in other spheres. We are proud of our famous engineers and now, when it seems that our greatest effort is required, we can perhaps draw guidance and inspiration from them. May I therefore in conclusion recall the words spoken by Joseph Locke, President of this Institution, at the meeting on the 8th November, 1859, after the deaths of Isambard Kingdom Brunel and Robert Stephenson:

"We at least, who are benefited by their successes, who feel that our Institution has reason to be proud of its association with such names as Brunel and Stephenson, have a duty to perform and that duty is to honour their memory and emulate their example."

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2. E. T. MacDermot, "History of the Great Western Railway," vol. 2. G.W.R., London, 1931.
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Mr V. A. M. Robertson, Past-President, moved

"That the best thanks of the Institution be accorded to the President for his Address and that he be asked to permit it to be printed in the Proceedings of the Institution."

In doing so, he remarked that it was most opportune that the President's impending retirement after 40 years' service with the Great Western Railway (and its successor—British Railways—Western Region) should coincide with his accession to the Presidential Chair of the Institution, because that evening he had given, and left to posterity, an Address which ranked as high as any Presidential Address which the Institution had ever received.

The Address had been divided into three parts. The first part dealt with Brunel and all he had done for the Great Western, including his famous design for the line from London to Bristol, his Maidenhead bridge, Box tunnel, Paddington station, and all those other works which had helped the Great Western, with its great traditions, and which had not been unhelpful to Mr Quartermaine in his own great task. The President had then dealt with the railways as he knew them to-day, and lastly with the railway civil engineer of to-day; and what he had said would not be disputed by any railway engineer present.

Towards the end of the Address the President had emphasized a point which Mr Robertson felt was very important. Any impression that the work of a railway engineer was lessening in interest and opportunities was wrong. There must be no room for doubt about the attractiveness of the railway service as a permanent career. In his farewell to the railways, that was the message which the President would leave behind. For 40 years he had faithfully grown up in and served the traditions of British railways, and at the end of his 40 years he had correctly stated in his Address to the Institution that the railway service must be traditional and must maintain its interest. That, it would be agreed, was the spirit in which a man would like to lay down a task undertaken with such success for 40 years.

Knowing the President as he did, having worked with him, and having watched him go from one step of the ladder to the next until he had reached the Presidential Chair of the Institution, Mr Robertson knew his keen interest, not only in what had gone behind but also in what lay in front of the Institution, and all that it meant to engineers. That would be uppermost in the President's mind during the next 12 months, and so, as a Past-President, Mr Robertson was confident that Mr Quartermaine, as President of the Institution, would inspire it with that enthusiasm and keenness which he had shown throughout his life and work, and that the Institution would be the better for it.

Mr J. A. Banks, who seconded the motion, said that he had listened enthralled to what had been an outstanding Address.

The President had certainly taken to heart the words of Robert Stephenson quoted early in the Address, and had observed keenly and comprehensively. No one listening to such a masterly review of Brunel's work and achievements could have other than a sense of humility in thinking of his own limitations, and yet be inspired to emulate if not to equal Brunel's efforts. The members must indeed be grateful to Mr Quartermaine for having given so much food for thought. They would feel that in him they had a President who would inspire them by his leadership and who would be a worthy occupant of the Chair which had been filled by so many eminent engineers.

The motion was carried with acclamation.

The President thanked Mr Robertson and Mr Banks for their remarks, which he said were much too good-natured. He wished also to thank all the members present for their kind reception of those remarks, and for their patient hearing. To the many who were there from the old Great Western Railway and who knew the early history of that railway he was deeply indebted for their silent endurance.

The kind remarks which Mr. Robertson and Mr. Banks had made, and the reception which had been given to them, was something which he would like to share with the members of his staff who had so readily produced information, photographs, and slides, and with his helpful and efficient secretary.

He would be very pleased if the Address were published in the Proceedings of the Institution.

MEDALS AND PREMIUMS, SESSION 1950–51

For Papers presented for discussion at Ordinary Meetings :—

(1) Coopers Hill War Memorial Medals and Prize to E. O. Measor, B.Sc.(Eng.), M.I.C.E. and D. H. New, B.Sc.(Eng.), M.I.C.E., joint Authors of the Paper on "The Design and Construction of the Royal Festival Hall, South Bank."
(2) A Telford Premium to Professor A. G. Pugsley, O.B.E., D.Sc. (Eng.), M.I.C.E., for his Paper on "Concepts of Safety in Structural Engineering."
(3) A Telford Premium to C. M. Roberts, M.I.C.E., for his Paper on "Fundamental Economics in Hydro-Electric Design."

For Papers presented at Meetings of the Engineering Divisions :—

ROAD ENGINEERING DIVISION

(1) A Telford Premium to R. D. Duncan, B.Sc., M.I.C.E., F. H. P. Williams, M.A., A.M.I.C.E., and the late J. S. K. Dalzell, B.Sc., A.M.I.C.E., jointly for their Paper on "A Full-Scale Experiment in Bog Blasting for Road Construction."

RAILWAY ENGINEERING DIVISION

(2) A Trevithick Premium to F. E. Campion, M.I.C.E., for his Paper on "Part Reconstruction of the Bo-Peep Tunnel at St. Leonards-on-Sea."
(3) A Trevithick Premium to C. W. King, M.I.C.E., for his Paper on "Arley Tunnel: Remedial Works following Subsidence."
(4) A Crampton Prize to John Graham, B.Sc., for his Paper on "The Reconstruction of Culvert No. 146, near Ayton, after its Destruction by Flooding in 1948."

STRUCTURAL AND BUILDING ENGINEERING DIVISION

(5) A Telford Premium to Professor A. L. L. Baker, B.Sc.Tech., M.I.C.E., for his Paper on "Recent Research in Reinforced Concrete, and its Application to Design."
(6) A Telford Premium to Gilbert Roberts, B.Sc.(Eng.), M.I.C.E., for his Paper on "The Structural Design of the 'Dome of Discovery,' Festival of Britain, 1951."
(7) A Trevithick Premium to G. P. Bridges for his Paper on "The Design and Construction of Silos and Bunkers."

WORKS CONSTRUCTION DIVISION

(8) A Telford Premium to A. E. Reid, B.Sc., M.I.C.E., and F. W. Sully, M.I.C.E., jointly for their Paper on "The Design and Construction of the Baghdad Railway and Road Bridge."

PUBLIC HEALTH ENGINEERING DIVISION

(9) A Telford Premium to B. A. Southgate, D.Sc., Ph.D., for his Paper on "Pollution of Streams: Some Notes on Recent Research."

For Papers published in the Journal with and without written discussion :—

(1) A Telford Premium to D. F. Denny, Ph.D., B.Sc.(Eng.), for his Paper on "Further Experiments on Wave Pressures."
2) A Telford Premium to B. G. Neal, M.A., Ph.D., and P. S. Symonds, Ph.D., jointly for their Papers on "The Calculation of Collapse Loads for Framed Structures," "The Calculation of Failure Loads on Plane Frames under Arbitrary Loading Programmes," and "A Method for Calculating the Failure Load for a Framed Structure subjected to Fluctuating Loads."

Professor Sutton Pippard, being a Member of Council, is ineligible to receive an award for his Paper on "Stresses in a Restrained Pipe-Line," and the Council have expressed to him the thanks of the Institution.

STUDENTS PAPERS

For Papers read before Local Associations and the Association of London Students :—

(1) The James Prescott Joule Medal and a Miller Prize to W. P. Hunter, B.Sc., Stud. I.C.E., for his Paper on " A Review of Current Practices in the Purification of Domestic Water Supplies " (West of Scotland Association).

(2) A Miller Prize to Arthur Marsland, B.Sc., Stud.I.C.E., for his Paper on " The Influence of Groundwater Flow on the Stability of Civil Engineering Structures and Earthworks " (Association of London Students).

(3) A Miller Prize to J. D. Geddes, B.Sc., and T. W. Weddell, B.Sc., Studs.I.C.E., jointly for their Paper on " Experiments on a Model of the Port of Sunderland " (Northern Counties Association).

(4) A Miller Prize to P. W. Agnew, B.A., Stud.I.C.E., for his Paper on " Telescopic Gangways at the New Passenger Terminal, Southampton Docks " (Southern Association).

(5) A Millar Prize to Robert McEwan, B.Sc., Stud.I.C.E. for his Paper on " Planning and Design Problems involved during Industrial Site Development " (East of Scotland Association).

(6) A Miller Prize to R. P. McCulloch, B.Sc., Stud.I.C.E., for his Paper on " The Construction of the Power Station Foundations at Grangemouth Oil Refinery " (East of Scotland Association).

(7) A Miller Prize to Peter Barnes, B.Sc., Stud.I.C.E., for his Paper on " The Mechanism of Railway Track " (Midland Counties Association).

(8) A Miller Prize to P. M. Worthington, B.Sc.(Eng.), Stud.I.C.E., for his Paper on " The Design of Economical Concrete Mixes " (South Wales and Monmouthshire Association).

(9) A Miller Prize to M. C. Purbrick, B.Sc.(Eng.), Stud.I.C.E., for his Paper on " Blanketing and Drainage of Railway Track, with Particular Reference to Work carried out at Parson Street, Bristol " (South Western Association).

(10) A Miller Prize to P. H. Steel, B.Sc., Stud.I.C.E., for his Paper on " The Reclamation of Land from the Sea " (South-Western Association).

THE INSTITUTION MEDAL AND PREMIUM

(Awarded annually to an undergraduate of the University of London for a Paper on engineering design, research, or practice)

Awarded to John Herbert Cuckney, Stud.I.C.E. for his Paper on " Impact on Railway Bridges."

BAYLISS PRIZES

(Awarded annually on the results of the Associate Membership Examinations)

Awarded to D. C. Musgrave (October 1950) and D. G. Llanwarne (May 1951).

Paper No. 5788

"Anchored Sheet-Pile Walls"

by

Peter Walter Rowe, Ph.D., A.M.I.C.E.

(*Ordered by the Council to be published with written discussion*) †

SYNOPSIS

Experiments with fifteen models of flexible walls retaining cohesionless soils are described, in which the influence of surcharge, anchor level, anchor yield, dredge level, soil type and state, pile flexibility, and distribution of soil pressure has been studied.

Agreement is reached with the experimental work of Stroyer, Browzin, and Tschebotarioff, under the same conditions of test. No vertical sand arching occurs on sheet-piling anchored to elastic supports. Decrease in bending moment for this case is due to fixity below the dredge line and to thrust between the anchor and the toe. The degree of fixity is determined by the flexibility of the pile and the relative density and compressibility of the soil. All previous theories and design proposals from experiments, which assume either fixed or free earth support, are correct for limited field conditions only.

A simple chart is proposed for the design of all types of anchored pile, for use with the "free earth support" analysis. Reinforced-concrete piling designed to low stresses is uneconomical in certain cases, whilst tidal conditions suggest the use of high-strength concrete or steel sections.

INTRODUCTION

ALL methods of anchored sheet-pile design conform with practical experience in that a reduction on the maximum free-earth-support value of the bending moment on the sheeting is allowed. The methods used may be broadly divided into two categories. The first, illustrated in *Figs 1* (*a*) and *1* (*c*), are those which assume Free Earth Support coupled with reduction in bending moment due to active pressure redistribution by soil-arching. The active and passive pressure distributions are calculated using Coulomb's theory, and the piling is driven to such a depth that only part of the passive earth pressure is mobilized. In this way, the piling is assumed to be freely supported by a single resultant force P_p near the toe, *Fig. 1* (*a*). The maximum bending moment acting with these forces is reduced either by an empirical redistribution[1] of the active earth pressure as in *Fig. 1* (*b*) or by the use of a reduction factor to be applied to the maximum bending moment,[2] (*Fig. 1* (*c*)). The second methods, illustrated in *Figs 1* (*d*) and *1* (*e*), assume fixed earth support. *Fig. 1* (*d*) shows the pressure distribution assumed to act on a pile driven deeper than that necessary for free support near the toe. The assumption is made that complete fixity will occur at depth d_1, and the penetration depth is calculated which conforms with the assumed forces.[3] This analysis leads to a smaller bending moment in the sheeting than that given by the Free Earth Support analysis, and further reduction is usually allowed owing to the possibility of arching. *Fig. 1* (*e*) shows the modified Fixed Earth Support method recently proposed by G. P. Tschebotarioff,[4] in which the maximum bending moment is assumed to be equal to that acting on a pile hinged at the dredge line.

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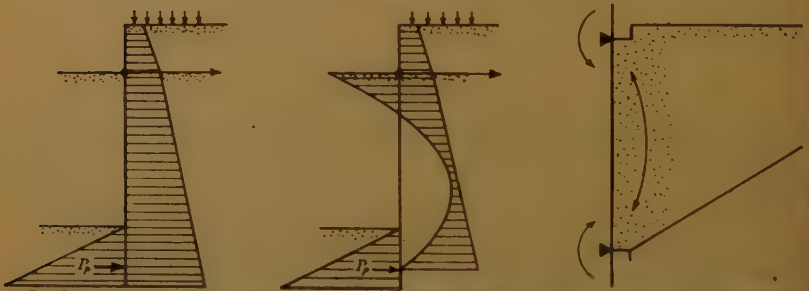
† Correspondence on this Paper should be received at the Institution by the 1st April, 1952, and will be published in Part I of the Proceedings. Contributions should be limited to about 600 words.—SEC. I.C.E.

[1] The references are given on p. 70.

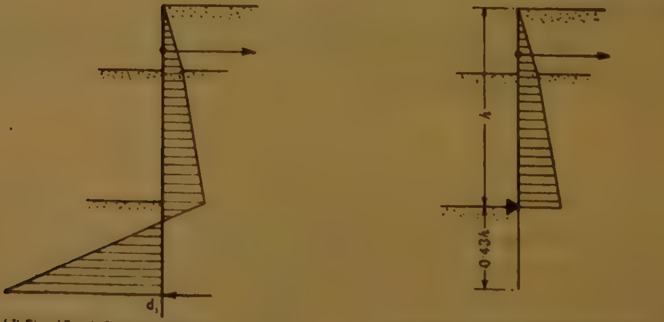
Figs 1



(*a*) Free Earth Support analysis

(*b*) Danish active pressure redistribution developed from practical experience

(*c*) J. P. R. N. Stroyer's measurement of moment reduction on a pinned plate due to soil arching



(*d*) Fixed Earth Support analysis: complete fixity assumed below d_1

(*e*) Prof. G. P. Tschebotarioff's simplified pile, hinged at the dredge-level

METHODS OF SHEET-PILE DESIGN

The sheet-pile structure may be divided into four types as shown in Table 1, depending upon the type of anchorage and the method of construction.

TABLE 1.—TYPES OF PILE STRUCTURE

Anchorage	Relieving platform	Elastic tie-rods
Method of construction	1. Fully driven and dredged	3. Dredged
	2. Partly driven and backfilled	4. Backfilled

G. P. Tschebotarioff, who determined the pressure distribution acting on a freely embedded model-pile indirectly from the bending-moment distribution, has recently shown that soil-arching only occurs with Type 1. Any reduction in bending moment on piling which is anchored by elastic tie-rods must therefore result from the condition of fixity below the dredge line.

The above methods have not taken into account the influence of the pile-section or the relative soil-density on the final behaviour of the structure. The models used by Tschebotarioff were arranged to simulate certain steel-piling already constructed under assumptions of fixity below the dredge line. The results, which confirmed these assumptions, apply only to the size and type of field pile studied, and do not prove that all sheet-pile structures should behave as if hinged at the dredge line. This is the source of the apparent contradiction between the above experiments and those of Boris S. Browzin.[5]

In view of the difference of opinion as to the value of indirect methods of measurement of soil pressures, a study of soil arching was first made using soil pressure-gauges.

NOTATION

c denotes concrete compression strength.
D ,, penetration depth.
D_r ,, relative soil density.
E ,, Young's Modulus.
e ,, void ratio.
f ,, design stress for steel piling.
G_p ,, factor of safety.
H ,, height of pile.
I ,, moment of inertia of section.
K ,, coefficient of volume compressibility.
K_a ,, active earth-pressure coefficient.
K_p ,, passive earth-pressure coefficient.
M ,, maximum positive bending moment.
m ,, bending moment.

P_a and P_p denote active and passive earth-pressure loads.

qs denotes surcharge pressure.

q	,,	surcharge coefficient: $q_s/\gamma H$.
q'	,,	equivalent coefficient for uniform water pressure.
R	,,	resultant passive load.
T	,,	tie-rod load.
T_s	,,	toe shear force.
t	,,	reinforcement design stress.
w	,,	density of water.
w_s	,,	weight of piling per square foot.
y	,,	pile deflexion.
z	,,	depth below surface.
α	,,	$1-D/H$.
β	,,	the ratio : $\dfrac{\text{depth to anchor-level}}{\text{pile height}}$.
γ	,,	earth density.
γ_w	,,	unit weight of earth submerged in water.
δ	,,	coefficient of friction between pile and soil.
ϵ	,,	τ/τ_{max}
η	,,	z/H.
ρ	,,	flexibility number H^4/EI.
τ	,,	M/H^3.
τ_{max}	,,	maximum value of τ for stiff piling.
ϕ	,,	coefficient of internal friction.

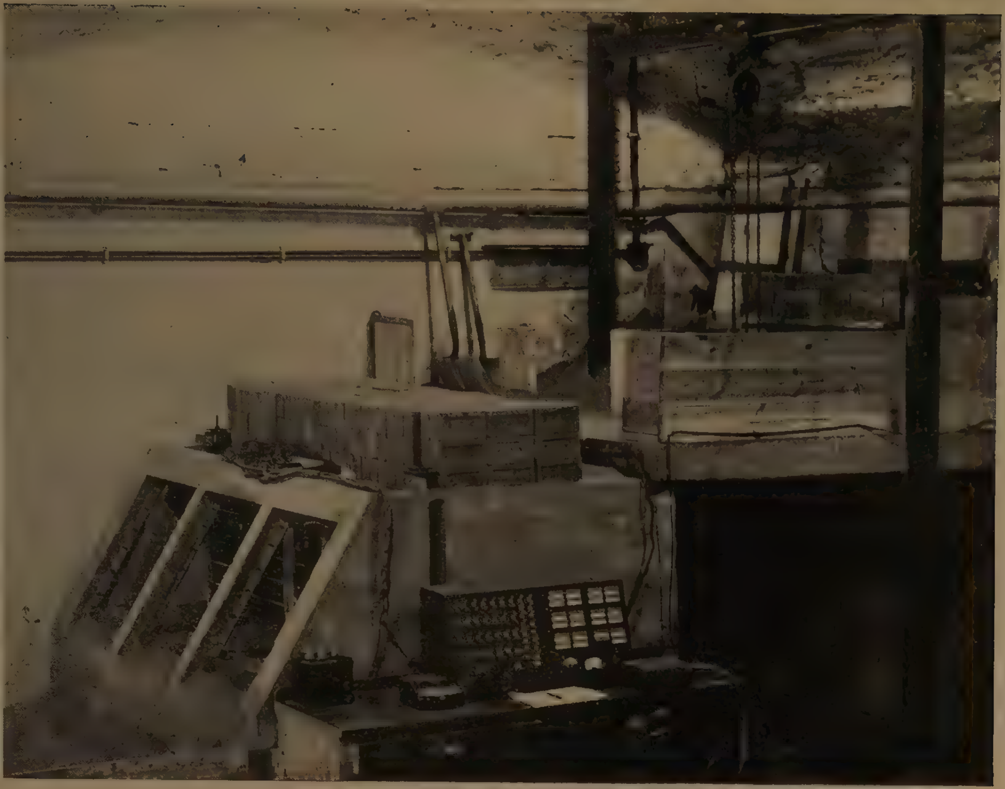
EARTH-PRESSURE DISTRIBUTION

Description of Apparatus

The apparatus is shown in *Figs 2* and *3*. The model-pile, which was 7 feet long by 3 feet 6 inches high, consisted of two 8-gauge plates bolted together along the centre-line and kept at a constant distance apart by wooden blocks. Forty-seven pressure-gauges[6] were mounted on the plates so that the diaphragms were flush with the outside wall, the gauges being between the plates. One-hundred-and-four strain-gauges were mounted on two vertical sections midway between the pressure-gauge sections, to measure the bending-moment distribution directly. Inside views of the model-pile are shown in *Figs 4* and *5*.

The shape of the reinforced-concrete bin necessary to hold the minimum volume of sand without affecting the behaviour of the model, was determined by separate experiments. The model was anchored to the rear of the bin by seven tie-rods connected in series with steel strips carrying strain gauges. The maximum elastic extension of the complete anchorage did not exceed 0·008 inch. The tie-rods could be released individually at the pile. Provision was made for four levels of anchorage. The yield at anchor level, vertical slip of the pile, and passive slip at the toe were

Fig. 3



General View of Apparatus Used

Fig. 4

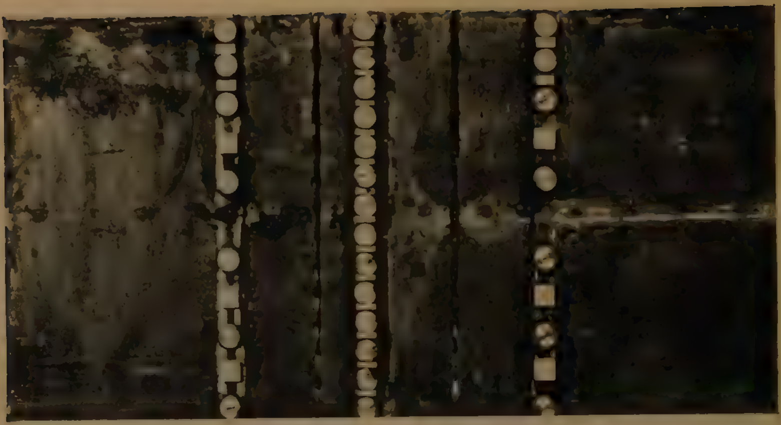


Plate facing Active Pressure

Fig. 5

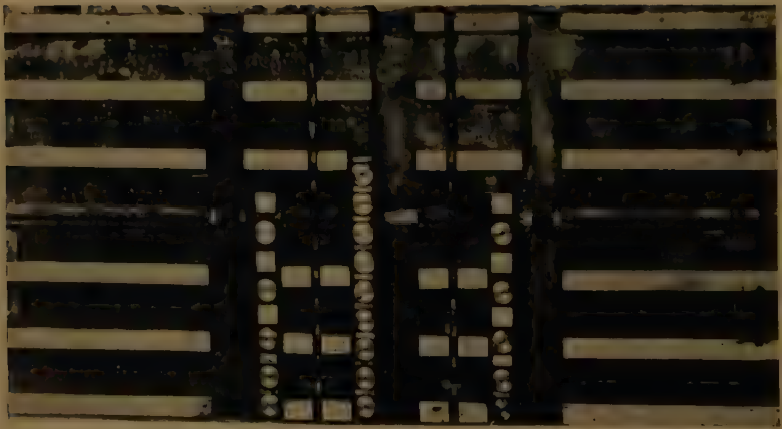


Plate facing Passive Pressure

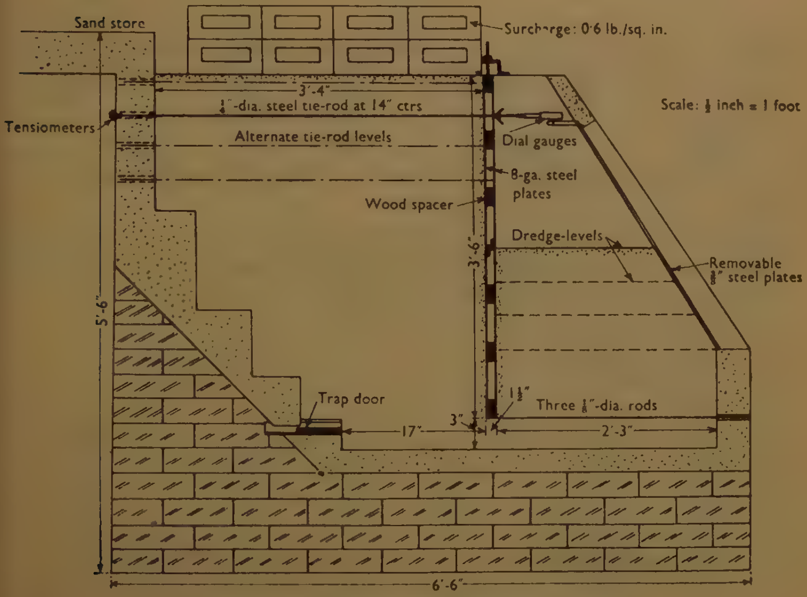
measured by dial-gauges reading to 0·001 inch. Engineering bricks were used to apply surcharge to the surface.

Calibrations

All the pressure gauges were calibrated six times during the course of the tests, by mounting a water-pressure chamber over each gauge in position on the pile.

The strain gauges were calibrated by testing the complete pile, with pressure gauges attached, in bending. The deflexion of the pile was measured over seven sections along its length, and small variations in the flexural rigidity of the model owing to the presence of the pressure gauges were eliminated by mounting steel plates, to represent dummy gauges, on the inside face of the pile. The zero pressure-gauge readings were recorded during the strain-gauge calibrations to confirm that the flexure of the pile did not affect the gauge readings.

Fig. 2



SECTION OF MODEL FOR PRESSURE TESTS

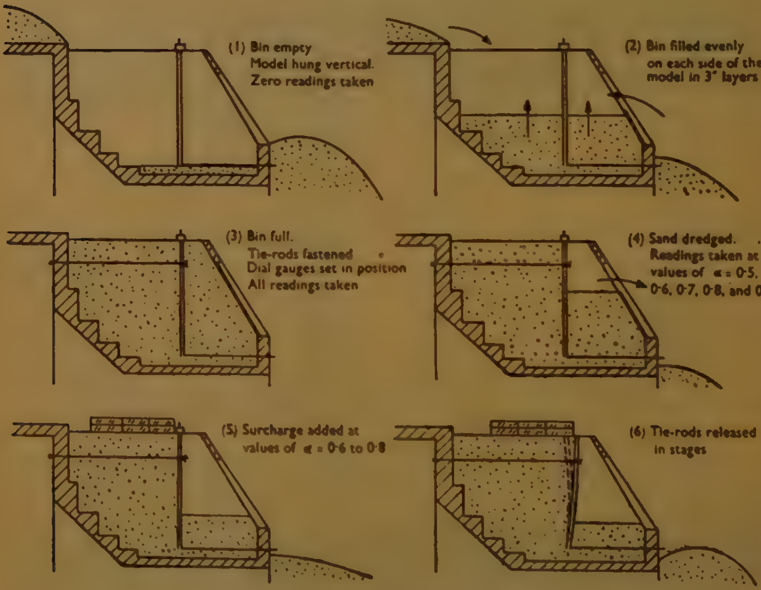
The tensiometers were calibrated by direct-loading tests. The mean weight of fifty of the engineering bricks was used to determine the surcharge pressure. The properties of the sand are shown in Table 3 and *Figs 11*, p. 40.

Method of Conducting Tests

A diagrammatic view of the general operations involved in each test is shown in *Figs* 6.

The pile was hung in a vertical position from supports at the side, and the zero readings of all the gauges were taken. The bin was filled with sand on each side of the pile, the sand level being raised evenly by the aid of guide-lines painted on both sides of the pile at 3-inch intervals. When the tie-rod level was reached, the tie-rods were placed in position and left loose. The filling was completed, the pile was released from its supports, and all gauge readings were taken. The tie-rods were set to be just tight.

Figs 6


(1) Bin empty
Model hung vertical.
Zero readings taken
(2) Bin filled evenly on each side of the model in 3" layers
(3) Bin full.
Tie-rods fastened
Dial gauges set in position
All readings taken
(4) Sand dredged.
Readings taken at values of α = 0·5, 0·6, 0·7, 0·8, and 0·9
(5) Surcharge added at values of α = 0·6 to 0·8
(6) Tie-rods released in stages

METHOD OF CONDUCTING TESTS

The sand on the outside of the pile was dredged away in stages by removing the steel plates at the front of the bin. All gauge readings were recorded at values of the dredged depth given by $\alpha=0{\cdot}5$, 0·6, 0·7, and 0·8. The dredging was either continued until complete passive slip occurred or it was stopped at an earlier stage and the effect of the addition of surcharge or tie-rod yield studied. When surcharge was added, the effect of the tie-rod yield was studied at the end of the test.

Method used to Check the Accuracy of the Observed Readings

The pressure distribution at each stage of dredging was used to calculate the bending-moment distribution and the mean tie-rod load. These values were then compared with those measured directly. An agreement in the maximum bending moment to within 10 per cent was considered satisfactory by allowing 5 per cent on the accuracy of the separate sets of measurements. In general, the agreement was of the order of 5 per cent for both tie-rod loads and bending moments, except where high surcharge pressure and large tie-rod yield combined to induce friction at the sides of the bin.

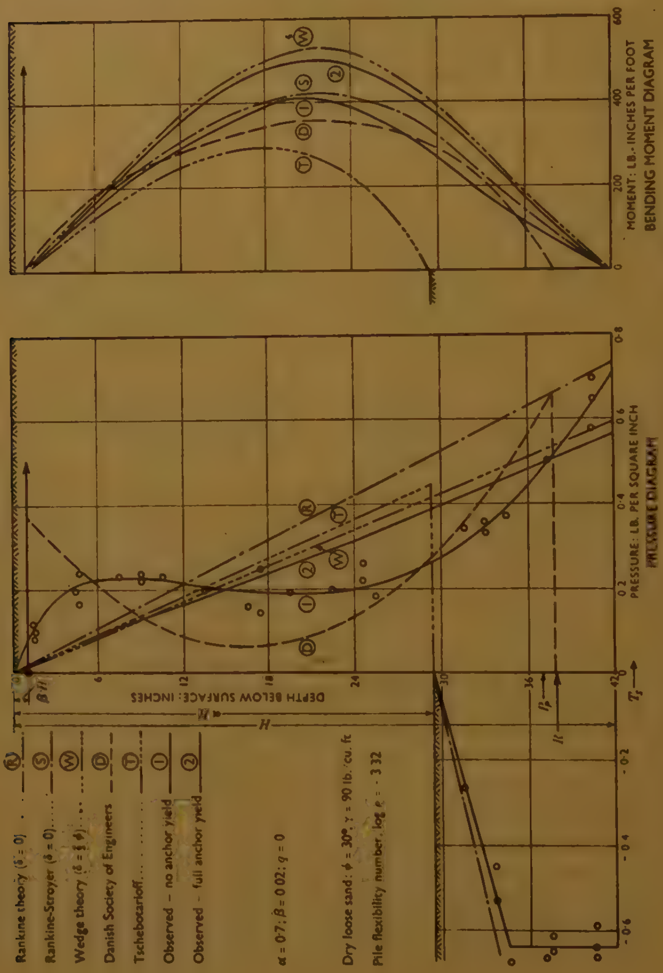
Discussion of the Result

Sixteen tests were carried out using four values of surcharge and four tie-rod levels for the conditions of no yield and full yield at the anchor. All these tests were confined to one model-pile and one dry sand in the loose state. A considerable amount of information was obtained from these tests and it is convenient to confine the discussion at this stage to the nature of the pressure distribution and the resultant bending-moment distribution. For this purpose, the examples shown in *Figs 7* and *8* are typical of all the tests. The following results were obtained :—

1. When the bin had just been filled, the distribution of active pressure was that given by the Coulomb theory with no wall-friction.
2. On dredging, with no anchor yield, the pressures increased at the tie-rod level and decreased at the centre. The total active load given by the area under the pressure diagram decreased, and at values of $\alpha = 0{\cdot}6$–$0{\cdot}8$ became equal to the Coulomb value assuming full wall-friction, that is, taking $\delta = \frac{2}{3}\phi$. The increase in pressure at the tie-rod level and decrease at the centre continued until passive slip occurred, provided that no yield of the anchor was allowed. At a safe value of the dredge level, the relief of pressure at the centre due to arching was considerably less than that assumed by the Danish Society of Engineers.
3. Outward yield of the tie-rods caused a breakdown of the arching and the pressure distribution became triangular, remaining equal in total area to the Coulomb value. The yield necessary to relieve completely the concentration at the tie-rod level varied with the amount of surcharge and the tie-rod depth, but a maximum value equal to $H/1{,}000$ was sufficient to destroy the arching in every case. This amount of yield is generally less than that which might be considered to occur in the field using elastic tie-rods anchored to plates subject to settlement. For example, if the tie-rods are of the same order of length as the height of the piling, then at 8 tons per square inch design stress, the elastic yield at the anchor is $\frac{8 \times H}{13{,}000}$ or approximately $\frac{H}{1{,}600}$. In

addition, the anchor plates must settle in the ground before their maximum passive resistance is developed. The total yield varies from $H/360$ for a 30-foot pile with 1-inch plate settlement to $H/930$ for a 70-foot pile with $\frac{1}{4}$-inch settlement.

Figs 7

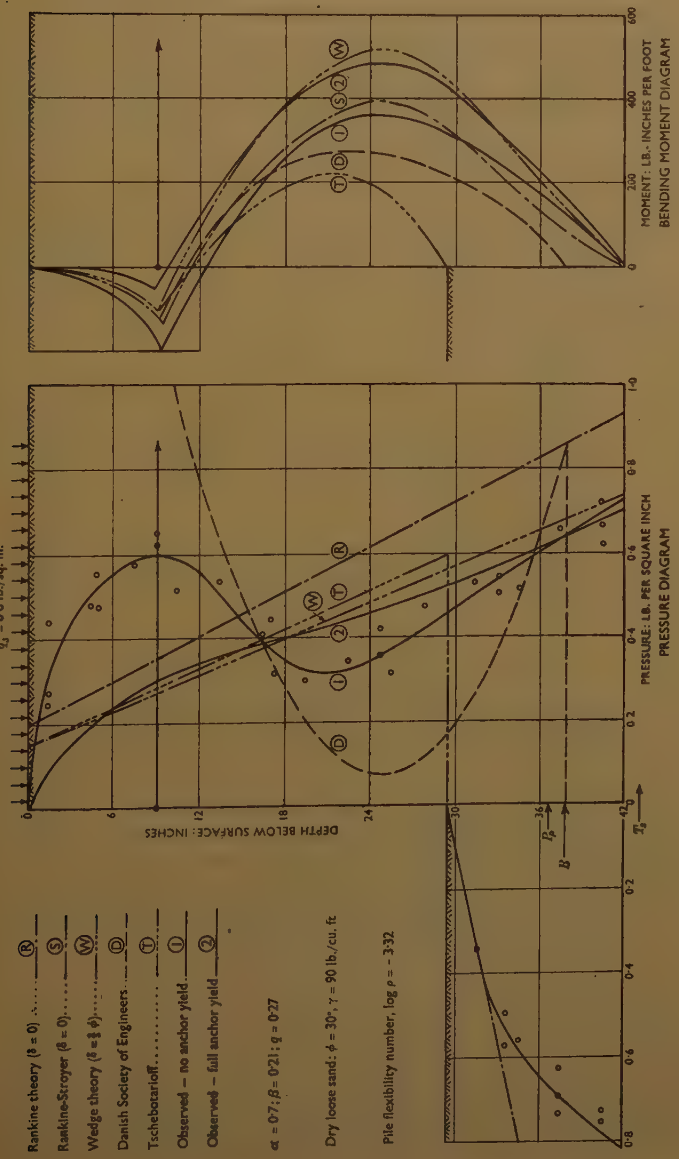


Typical Result for Anchorage near Surface. No Surcharge

4. The passive pressures at the dredge line increased with dredging, from the active pressure value at $\alpha=0$ to the Coulomb passive value $(\delta=0)$ when α was approximately equal to 0·7. Only part of the complete triangular pressure-distribution was mobilized at this stage and the wall movements were very small. With further

dredging, the movements at the toe increased in size, developing wall-friction on the passive side. Ultimate failure occurred when the Coulomb-value with full wall-friction was fully mobilized.

Fig. 8

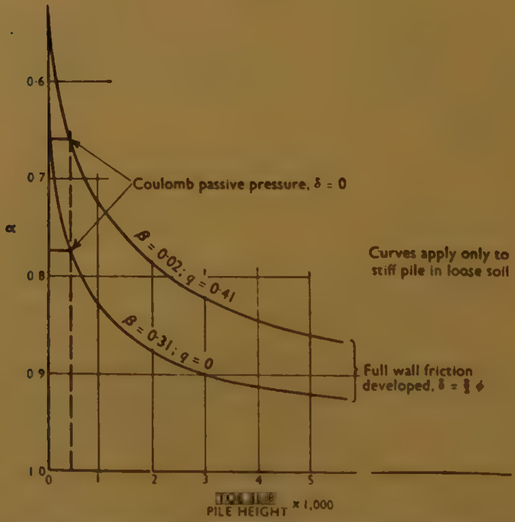


TYPICAL RESULT FOR DEEPER ANCHORAGE. SURCHARGE

The order of the wall movements at the toe, developed by dredging, are shown in *Fig. 9.*

5. During slip a considerable shear force acted between the toe of the pile and the subsoil.
6. Before yield, the maximum bending moment agreed closely with that calculated from Stroyer's empirical formula. That was to be expected because the model behaved as if pinned at the tie-rod level and at a distance from the toe equal to one-third of the penetration depth, but it is interesting to note that the agreement held for all tests with uniform surcharge, whereas surcharge was not used in Stroyer's experiments.

Fig. 9



WALL MOVEMENTS AT TOE, DEVELOPED BY DREDGING

7. Upon yield, the bending moment increased to that given by the Coulomb theory. The maximum bending moments at this stage were always larger than those recommended by Tschebotarioff—in some cases more than twice the value.

Conclusions

The active pressure distribution acting on a sheet-pile structure anchored to yielding supports is triangular and equal to the Coulomb value with full wall-friction. This confirms the conclusion of Tschebotarioff. The difference in the value of the maximum bending moment acting on yield with that determined by Tschebotarioff constitutes a fundamental difference in the behaviour of two models even though they be subjected to the same active pressure distribution.

The difference may be resolved by the study of the influence of the flexibility of the pile on its behaviour. The apparatus and experiments on this subject are described in the following section.

FLEXIBILITY TESTS

In order to study the influence of pile flexibility on the factors governing design, repeated tests over a range of flexibilities were required on models having the same values of tie-rod level coefficient β and surcharge coefficient q for the condition of yield. This was achieved by varying the height, the moment of inertia, and the Young's Modulus of the model.

For similarity between model and prototype,

$$\left(\frac{dy}{dz}\right)_{\text{model}} = \left(\frac{dy}{dz}\right)_{\text{prototype}} \qquad \text{therefore} \quad \left(\frac{Mz}{EI}\right)_m = \left(\frac{Mz}{EI}\right)_p$$

If surcharge $q_s = q\gamma H$, and $\dfrac{z_m}{H_m} = \dfrac{z_p}{H_p} = \eta$

then, for given values of β, q, and α,

$$\frac{M_m}{{H_m}^3} = \frac{M_p}{{H_p}^3} = \tau$$

Therefore

$$\left[\frac{H^3 \tau H \eta}{EI}\right]_m = \left[\frac{H^3 \tau H \eta}{EI}\right]_p$$

that is to say,

$$\frac{H^4}{EI} = \text{constant} = \rho$$

A field-pile deflexion will therefore be similar to that of a model for the same values of β, q, α, and soil properties, if the values of the flexibility numbers ρ are equal.

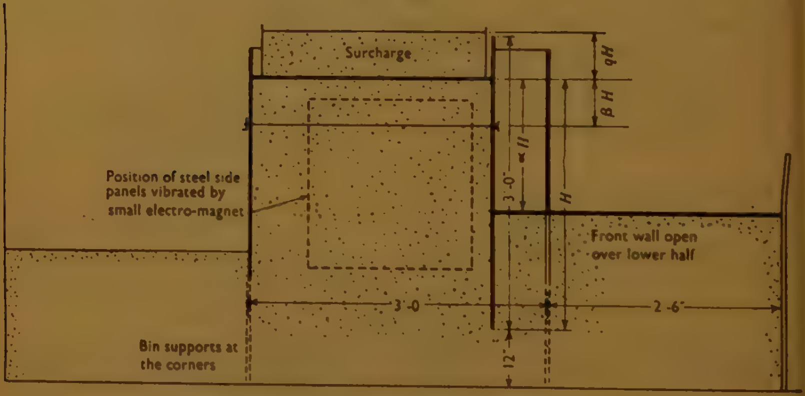
Description of Apparatus

The apparatus is shown in *Fig. 10.* The bin was 3 feet square in plan and 2 feet 6 inches deep. It had no floor but rested on a 12-inch bed of sand. The front was open over the lower portion allowing the "passive" sand to extend 3 feet from the model.

The height of the model tested varied from 20 inches to 36 inches. In the case of the 20-inch model, the length of the bin was 1·8 times the height of the model, whilst this ratio decreased to 1 : 1 for the 36-inch model. At this ratio, using surcharge, some side friction would be expected to occur. In order to avoid using a longer bin, the problem of reducing side friction was studied experimentally. As a result, the wooden sides of the bin were partly replaced by sheets of steel so that these sheets were adjacent to the volume of the retained soil within the slip surface. These sheets were vibrated using a small electro-magnet at high frequency, and with small-amplitude vibrations. This was found to destroy the side friction by creating a highly localized state of flux adjacent to the bin wall. Measurement of the height of the sand in the bin before and after vibration showed that the compaction due to the vibration only occurred to a small degree immediately adjacent to the sides of the bin. Details of the model-piles used are given in Table 2.

The model scales and plate sizes were chosen to space the flexibility number ρ at regular intervals on a logarithmic scale. Provision was made for attaching the tie-rods at depths below the surface consistent with values of β equal to 0, 0·1, 0·2, and 0·3, at all the model-scales.

Fig. 10


Surcharge
Position of steel side panels vibrated by small electro-magnet
Bin supports at the corners
3'-0"
3'-0"
2'-6"
12"
qH
βH
H
Front wall open over lower half

APPARATUS FOR FLEXIBILITY TESTS

Surcharge was applied by placing a thin wooden frame on the soil surface and filling inside with the soil. The surcharge was always proportional to the scale according to the relation $q_s = q\gamma H$. Details of all soils used are given in Table 3 and *Figs 11*.

Method of Conducting Tests

The tests were confined to the case of tie-rod yield. For this case, no difference exists between the " driven and dredged " and the " backfilled " pile.

The bin was filled evenly on both sides of the model until three-quarters of the model was embedded in the sand, when the initial strain-gauge readings were taken. The tie-rods were set, the filling completed at the rear, and the soil dredged on the outside to the first value α=0·6. The strain gauge which was likely to be recording the maximum bending moment was then switched in, and the galvanometer was allowed time to steady.

TABLE 2.—DETAILS OF MODEL-PILES

Material	Plate gauge : S.W.G.	Height : inches	Log H^4/EI	Test
Steel	10	30	$-3 \cdot 22$	Preliminary tests to determine shape of the R.C. bin.
	18	20	$-2 \cdot 72$	
„	Double wall 8	42	$-3 \cdot 32$	Pressure tests.
„	8	36 32 30 28 26	$-3 \cdot 18$ $-3 \cdot 38$ $-3 \cdot 49$ $-3 \cdot 61$ $-3 \cdot 74$	Flexibility tests.
„	12	36 31·5 27·5 24 21	$-2 \cdot 52$ $-2 \cdot 74$ $-2 \cdot 98$ $-3 \cdot 22$ $-3 \cdot 45$	
Duralumin	14	29 26 23 20	$-2 \cdot 07$ $-2 \cdot 26$ $-2 \cdot 48$ $-2 \cdot 72$	

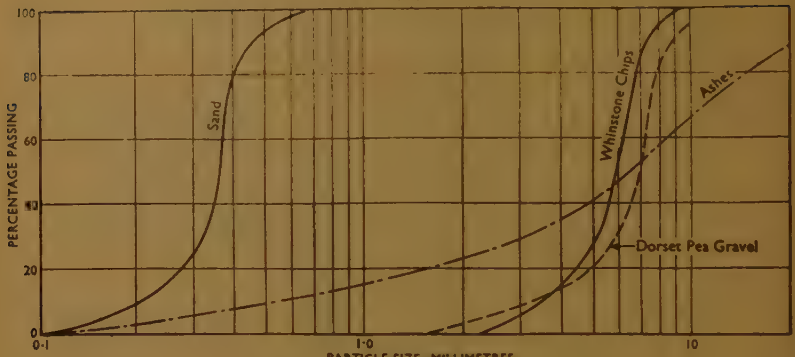
The strain gauges on the models used for the flexibility tests were spaced at 2½-inch intervals on two sections on both sides of each model.

TABLE 3.—SOIL PROPERTIES

Soil	Density : lb./cu. ft.	ϕ : degrees	Void ratio	K : in²/lb. $\times 10^{-3}$	Relative density
Sand	90	30	0·78	3·0	0·0
Dorset Pea Gravel	98	30	0·74		
Whinstone Chips .	82	39	1·06		
Ashes	40	40	1·76	15	
Sand	100	41	0·53	0·6	1·0
Dorset Pea Gravel	110	37	0·49		
Whinstone Chips .	96	50	0·74		
Ashes	56	50	0·95		

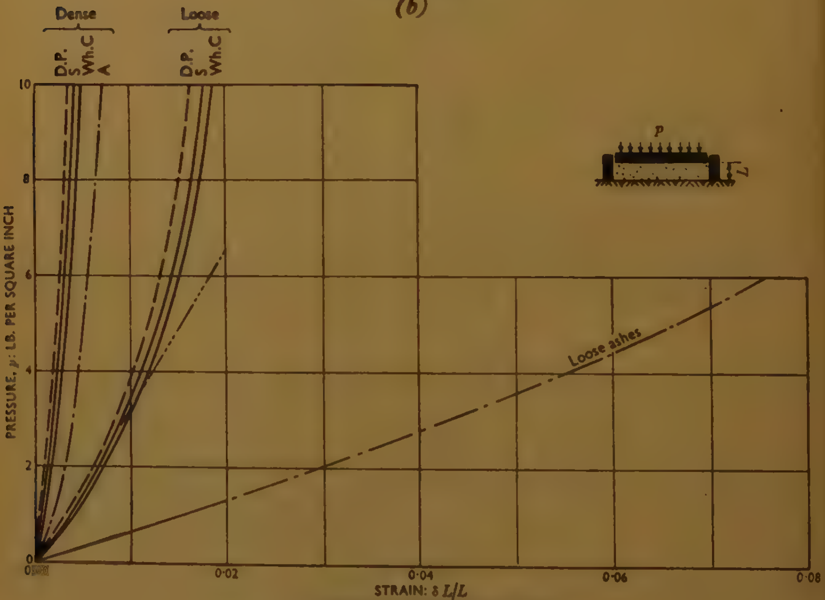
Figs 11

(a)



SIEVE ANALYSIS

(b)



CONFINED COMPRESSION CURVES

Details of Soils used in Tests

The anchorage was then released. While vertical sand-arching was being destroyed, the bending moment increased. However, too much yield eventually caused a decrease, owing to cantilever action from the dredge level. Accordingly, the anchorage was released carefully until the galvanometer indicated maximum deflexion. The increment of yield required after each stage of dredging was approximately $H/2,400$, giving a total yield of $H/800$ at the end of the test.

The sides were next vibrated and the galvanometer reading increased to a further amount varying from 0 to 10 per cent according to the type of test. All gauge readings were then recorded and the dredging was continued.

Tests with soil in the dense state were made by compacting the soil in 1-inch layers on either side of the pile with a high-frequency hand vibrator.

Range of Tests

Tests were made using ten values of pile flexibility with four values of tie-rod level and three values of surcharge. The above tests were repeated for four types of soil in the loose and dense states.

Discussion of the Results. Bending Moments

Loose Soil

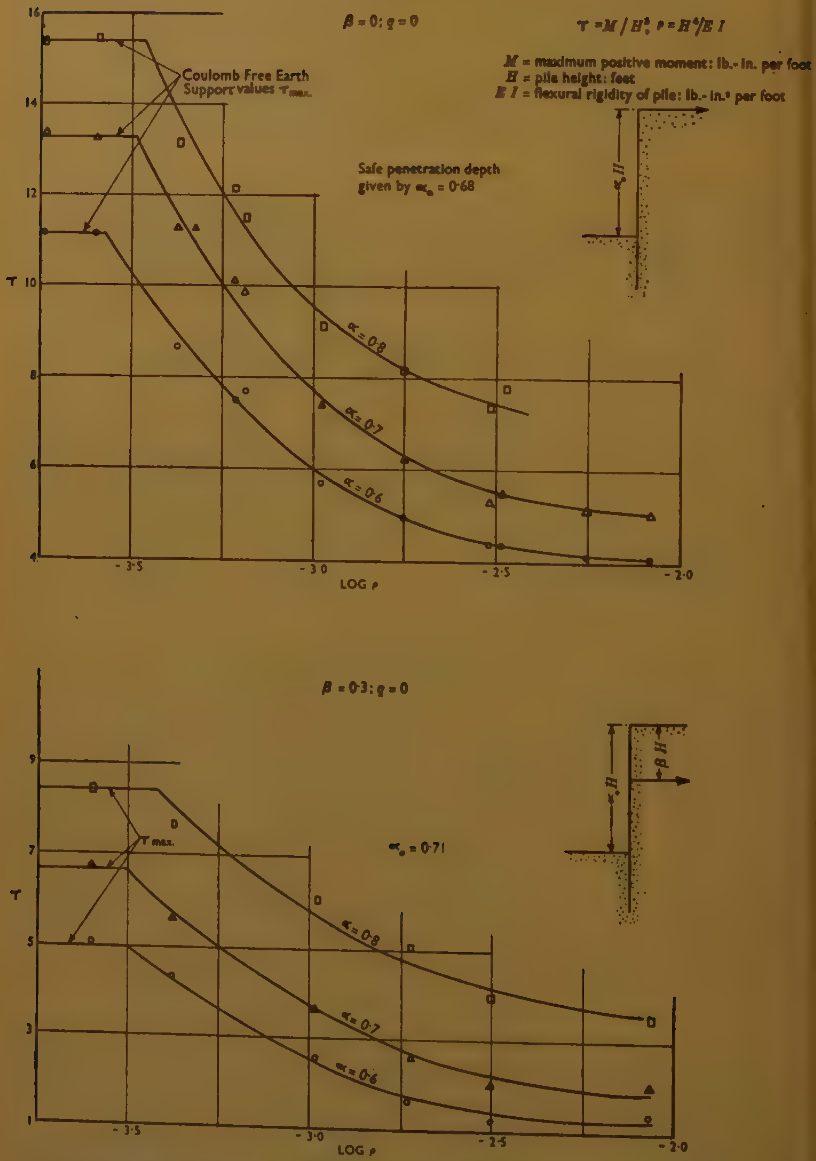
Four examples of tests with loose sand are shown in *Figs 12* where the maximum positive bending moments per unit model-height τ are plotted against the flexibility numbers log ρ for three values of the dredge level in each case. These results are replotted in *Figs 13*, the maximum moments being recorded as a percentage of the maximum value for a stiff pile, having log $\rho < -3{\cdot}5$.

The graphs in *Figs 13* show that points obtained from tests with varying values of β and q lie, for practical purposes, on a mean curve at each value of α. The mean curves at values of $\alpha = 0{\cdot}6 - 0{\cdot}8$ are plotted in *Fig. 13* (*d*) and these show only a small variation with dredge level. The majority of structures have been designed with penetration depth corresponding to values of α between 0·65 and 0·75, so that the mean result for α equal to 0·7 may be taken as representative for all the tests with loose sand.

The graphs obtained for the other soils in the loose state showed similar mean curves for all values of β and q. The mean curves for $\alpha = 0{\cdot}7$ are shown in *Fig. 14*. The curves are identical for three of the soils used having widely varying characteristics, and a mean curve has therefore been drawn. These curves show that the percentage decrease in bending moment on sheet-piling subject to tie-rod yield is mainly a function of pile flexibility and soil relative-density and is independent of the values of β, q, ϕ, and γ.

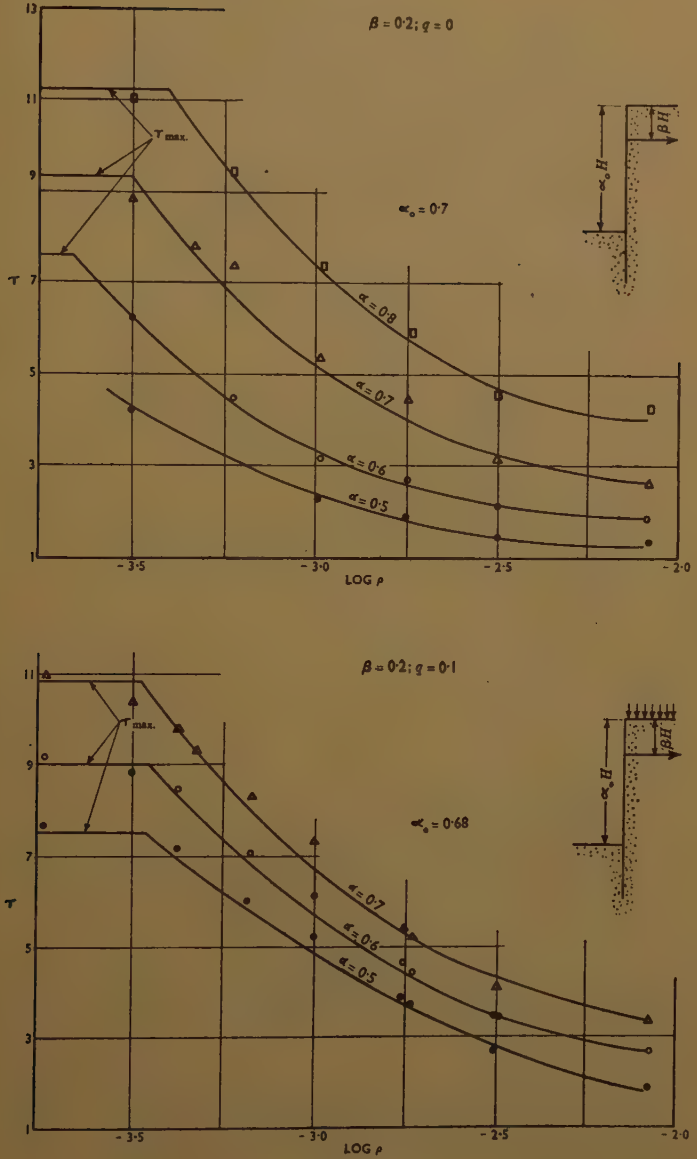
The reason for the decrease of bending moment is shown diagrammatically in *Fig. 15* (*d*). A$_1$B represents a stiff pile in loose sand having log ρ less than $-3{\cdot}5$. The deflexion at the dredge line is very small and of the same order as small movements at the toe. This results in an approximately horizontal movement of the sheeting below the dredge level into the soil. This is consistent with the development of the triangular type of pressure distribution observed in the first experiments and the resultant passive force acts at a point one-third of the dredged

Figs 12 (a)



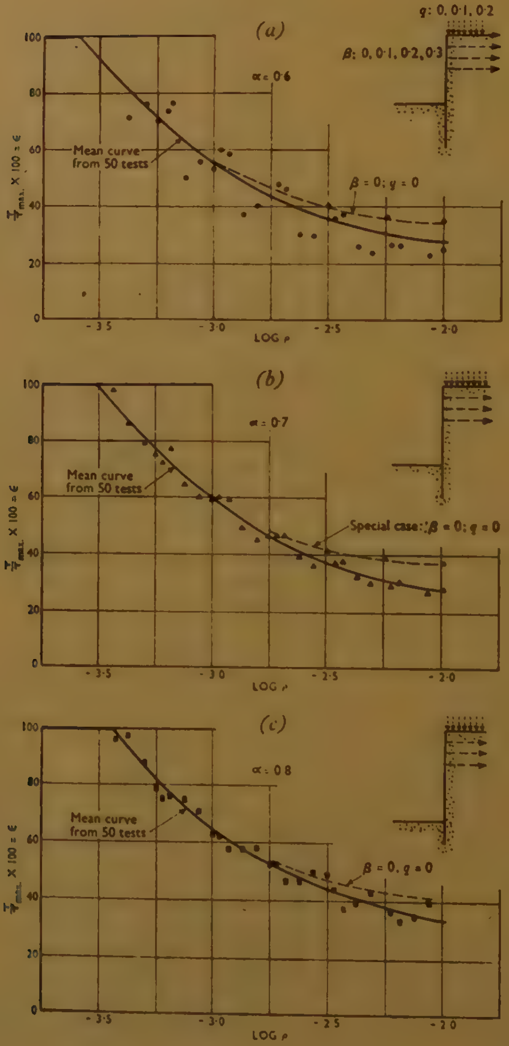
Two Examples of Bending-Moment Reduction with Increase in Pile Flexibility. Full Anchor Yield. Tests made with Loose Dry Sand

Figs 12 (b)



Two Further Examples of Bending-Moment Reduction with Increase in Pile Flexibility. Full Anchor Yield. Tests made with Loose Dry Sand

Figs 13 (a), (b), and (c)

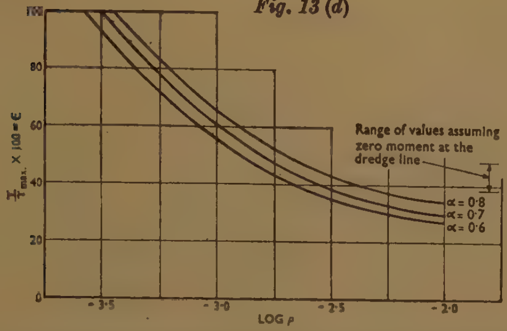


τ = observed max. pos. B.M. per unit of model height

$\tau_{max.}$ = Free Earth Support value of τ for each particular value of β, q, and α, taking $\delta = \frac{2}{3}\phi$

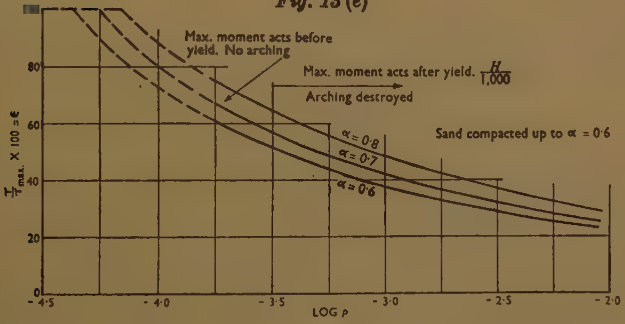
MEAN PERCENTAGE BENDING-MOMENT/FLEXIBILITY CURVES FOR LOOSE SAND
Anchor Yield

Fig. 13 (d)



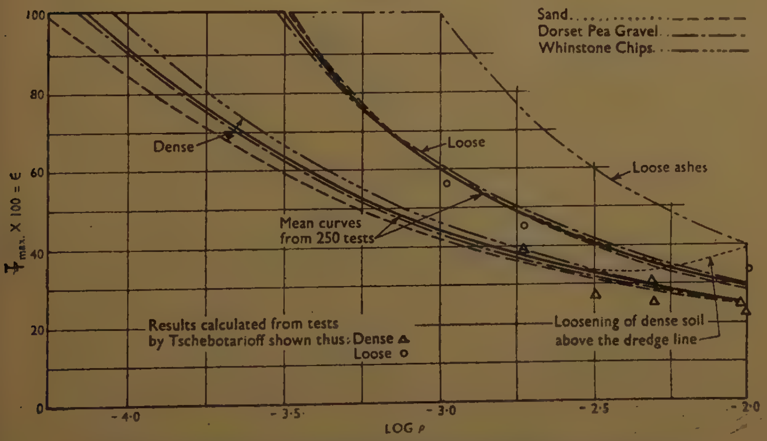
MEAN CURVES FROM *Figs 13* (*a*), (*b*), AND (*c*), REPLOTTED

Fig. 13 (e)



MEAN PERCENTAGE BENDING-MOMENT/FLEXIBILITY CURVES FOR DENSE SAND

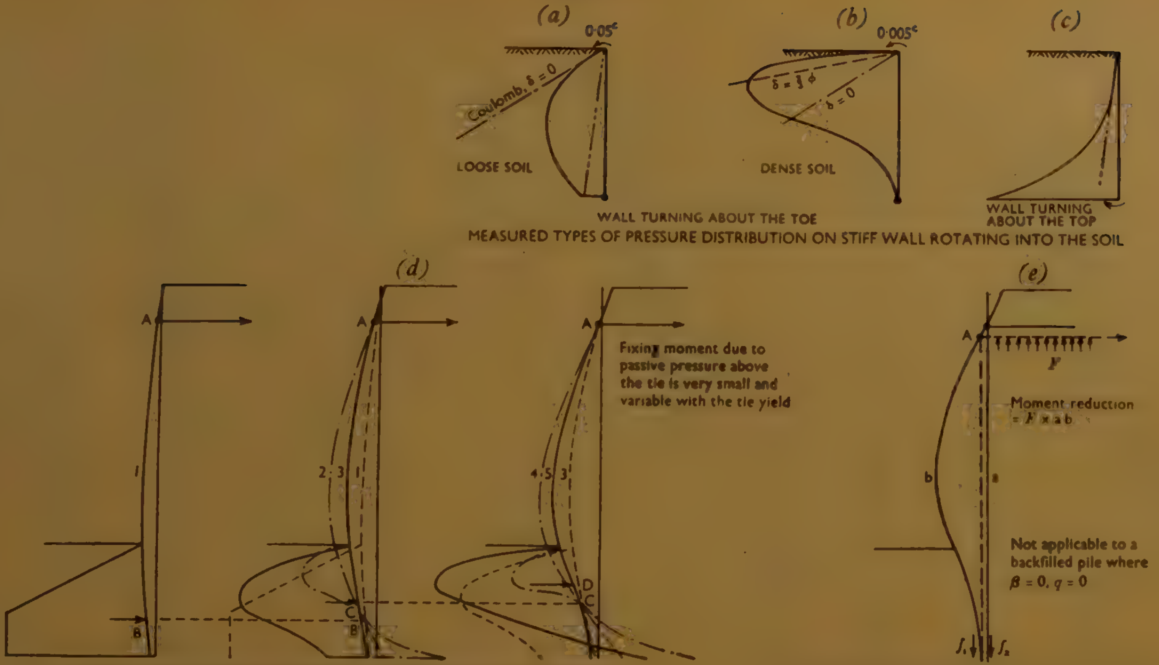
Fig. 14



MEAN PERCENTAGE BENDING-MOMENT/FLEXIBILITY CURVES

Anchor Yield $> \frac{H}{1,000}$

Figs 15


(a)
0·05ᶜ
Coulomb, δ = 0
LOOSE SOIL
(b)
0·005ᶜ
δ = ⅔ φ
δ = 0
DENSE SOIL
(c)
WALL TURNING ABOUT THE TOE
WALL TURNING ABOUT THE TOP
(d)
A
1
B
A
2·3
C
B
A
4·5
3
D
C
Fixing moment due to passive pressure above the tie is very small and variable with the tie yield
(e)
A
F
Moment reduction = F × a b
b
a
Not applicable to a backfilled pile where β = 0, q = 0
f₁
f₂

MEASURED TYPES OF PRESSURE DISTRIBUTION ON STIFF WALL ROTATING INTO THE SOIL

CHANGE IN PASSIVE PRESSURE DISTRIBUTION WITH INCREASE IN PILE FLEXIBILITY

BENDING MOMENT REDUCTION DUE TO END THRUST

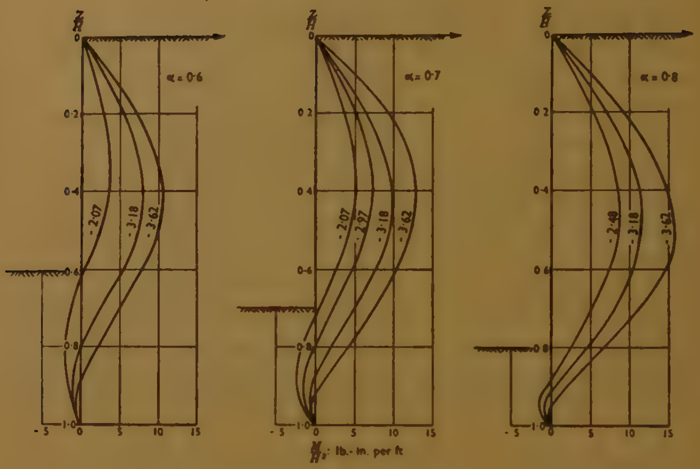
TWO MAIN CAUSES OF BENDING MOMENT DECREASE WITH INCREASE IN PILE FLEXIBILITY depth above the toe. If the pile is replaced with one of greater flexibility it would normally tend to deflect further to A2B. This involves an additional rotational movement about B into the passive soil and this movement is consistent with a passive pressure distribution which is a complex function of depth and deflexion, and the soil-density. Direct measurement of the pressure distribution resulting from a stiff plate rotating into the soil, by means of the pressure gauges, gave distributions as shown in *Figs 15* (*a*) and (*b*). The centre of pressure of the resultant distribution, being higher than B, *Figs 15* (*d*), causes the point of zero bending moment on the pile to rise to C, and the pile to deflect to A3C. By replacing this pile with another of higher flexibility, further deflexion by rotation will occur at first about C, which, in turn, raises the centre of pressure of the passive pressure diagram to D. The direct measurements of pressure showed that very high passive pressures can be sustained near the dredge line. The maximum height reached by the point of zero bending moment for the case of the highest flexibility used, conformed to that calculated by the Fixed Earth Support method, in the case of loose sand.

Since the bending moment varies as the cube of the span, a large decrease in bending moment occurs with a small rise in the point of the resultant passive pressure. To a certain extent, this has meant that the lighter the section used, the smaller the bending moment it has had to withstand. This factor, which has nothing whatever to do with arching on the active pressure side of the wall, has been the main cause of the reduced deflexion and the stability of walls anchored to elastic supports.

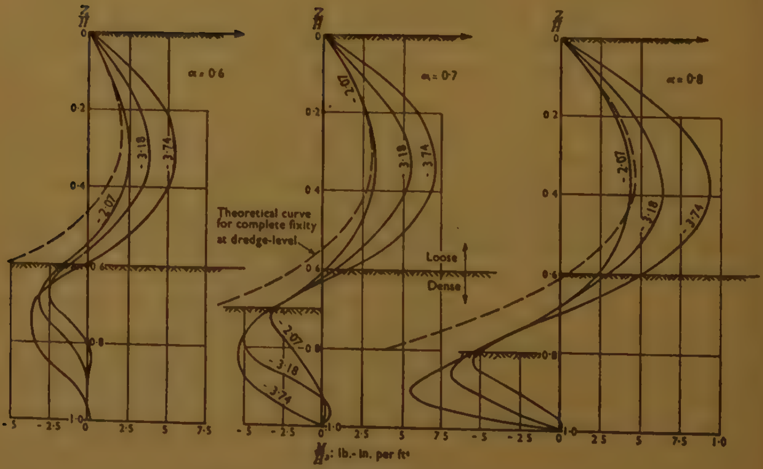
In addition, there is a secondary source of moment reduction. It will be seen from *Fig. 13* (*d*) that the minimum recorded bending moments are less than those calculated, assuming that the point of zero bending moment had reached the dredge level. This leads to another source of bending moment decrease. In *Fig. 15* (*e*), the flexible pile in deflecting, changes shape from AaB to AbB, and requires that the ends A and B move to-together. This is negligible in the case of the stiff piling but not so with the flexible piling. The free movements of the ends are resisted by shear force F balanced by the bearing pressure of the tie-rods on the retained soil. This causes a further reduction in moment approximately equal to $F \times$ ab. Measurements showed that the necessary decrease in span AB exceeded the settlement of the tie-rods in the case of the flexible models and that the maximum possible value of the shear force F was sufficient to cause the further decrease in moment.

This cannot apply to the particular case where the pile is backfilled and $\beta = q = 0$. The results for this case gave higher minimum bending moments for the flexible pile, which were close to the Fixed Earth Support value. With deep tie-rods, passive pressure above the ties may occur with flexible piling even with tie-rod yield. These factors influence to a small degree the position of the points in *Figs 12*.

Figs 16 (a)



SAND, LOOSE: $\beta=0$, $q=0$

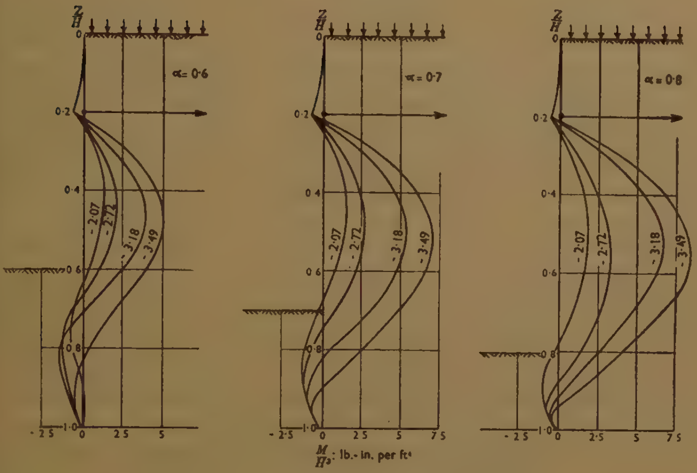


SAND COMPACTED BELOW DEPTH $0{\cdot}6H$

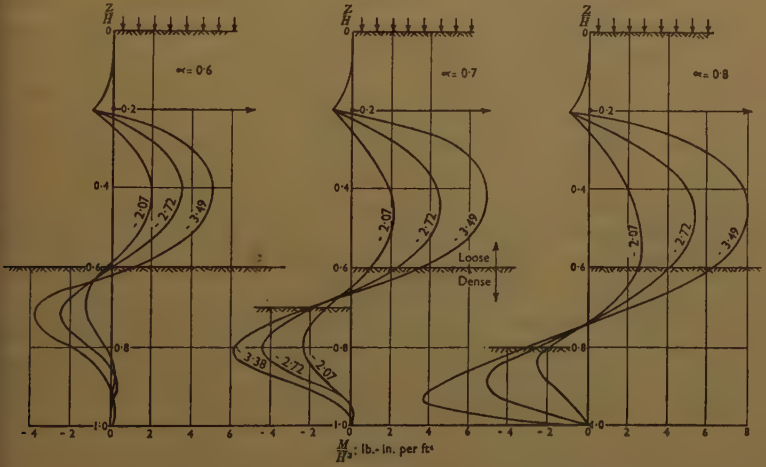
$\beta=0$, $q=0$

BENDING-MOMENT DISTRIBUTIONS

Figs 16 (b)



WHINSTONE CHIPS, LOOSE: $\beta=0{\cdot}2$ $q=0{\cdot}1$



DORSET PEA GRAVEL COMPACTED BELOW DEPTH $0{\cdot}6H$

$\beta=0{\cdot}2$, $q=0{\cdot}1$

BENDING-MOMENT DISTRIBUTIONS

Soil Compacted below the Dredge Line

The curves corresponding to *Fig. 13* (*d*) for loose sand are drawn in *Fig. 13* (*e*) for sand compacted to its maximum density. The mean curve at $\alpha=0{\cdot}7$ has been taken as representative for all the tests and this is plotted in *Fig. 14* together with the equivalent curves for the other soils. A difference occurs between the mean curves for the different soils, but this may be due to small inaccuracies in the measurement of the angle of friction of the compacted soil. A mean curve has therefore been drawn which is similar in form to that of the loose soils. In addition to the main reasons for the moment decrease given for loose soils, two further factors affect the shape of the curve. It is seen from *Fig. 15* (*b*) that the centre of passive pressure acting on a stiff plate rotating into the dense soil about a point below the surface, is higher than that for loose soil, and the corresponding angular wall movements are smaller. Consequently, when the decrease in moment begins, a much smaller decrease in stiffness, or rotation, is necessary to complete the full reduction from this cause. While the point of zero bending moment is rapidly raised above the dredge line at quite low values of ρ, the maximum moment on the sheeting does not fall more rapidly than for the loose soil as might otherwise be expected. This is partly due to the lack of end-thrust at this stage and partly due to a decrease in wall-friction. The decrease in wall-friction occurs since stiff walls rigidly embedded in dense soil do not bend or move away from the soil sufficiently for its development.

The exact location of the moment-reduction/flexibility curve for the dense soils for flexibilities given by $\log \rho<-3{\cdot}5$ is influenced by the degree of tie-rod yield. With no yield, moments due to earth pressures at rest were recorded. With yield, the moment decrease was continuous owing to reduction in active pressure and cantilever action of the piling. All the moments recorded were maximum values. For the bulk of the tests, on loose soils, and dense soils with pile flexibility given by $\log \rho>-3{\cdot}5$, these values occurred after tie-rod yield, equal to $H/1{,}000$, had destroyed arching. For the few tests with dense soils and $\log \rho<-3{\cdot}5$, the pile deflexions were so small that no arching was present, and the maximum moments occurred with yield $<H/1{,}000$. For these reasons, the curves in *Fig. 13* (*e*) for the dense soils have been extrapolated from $\epsilon=70$-per-cent to the 100-per-cent line. It will be seen later that this portion of the curves lies well outside the region of practical design.

Typical moment distributions are shown in *Figs 16.*

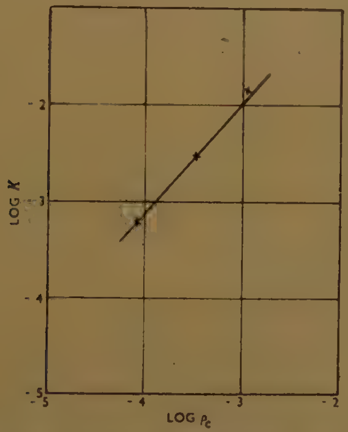
Retained Soil Compacted to the Surface

The percentage moment/flexibility curves are the same as for soil compacted to the dredge level, as the reduction is governed by the soil condition below the dredge level and is independent of the total active moment. However, when a pile having a flexibility in excess of the value $\log \rho=-2{\cdot}5$ is used, the deflexion of the piling is sufficient to loosen the soil above the dredge line. This causes the bending moment to increase, as shown in broken line in *Fig. 14*.

Critical Flexibility

The value of ρ at which the bending moment commences to fall below the Free Earth value, will be called the critical flexibility ρ_c. It is reached when the deflexion at the dredge line equals the deflexion at the toe, so that the value of ρ_c should be related to the coefficient of volume compressibility for the soil. Confined compression tests were made on the soils using pressures within the range likely to occur in practice, as shown in *Fig. 11* (*b*). The initial slope of the curves has been used to calculate a mean coefficient of volume compressibility K for the soils in the dense state, the sand and gravels in the loose state, and the ashes. The values of ρ_c and K have been plotted on a logarithmic base in *Fig. 17*, where it is seen that within the limits of the range of the experiments, ρ_c is approximately proportional to K. Since the passive pressures on the prototype exceed those on the model, the effective compressibility coefficient will be less and the value of ρ_c smaller in the field than in the laboratory. This provides a small margin of safety.

Fig. 17



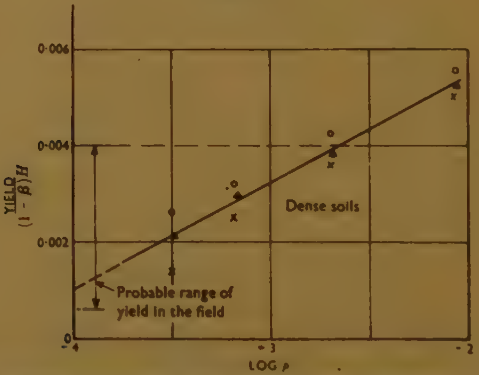
Approximate Relation between Compressibility of Soil and Critical Flexibility of Pile

Where the subsoil consists of clean sand or gravel, a measurement of the relative density will enable the value of ρ_c to be estimated and the moment/flexibility curve to be obtained by interpolation. A high percentage of mica in the sand or organic matter may increase the compressibility and, in this event, confined compression tests should be made.

Effect of Tie-Rod Yield on the Maximum Negative Bending Moments

Tie-rod yield causes an increase in the maximum negative moment below the dredge line. The yield necessary to cause the moment to exceed the values given in *Fig. 14* for the positive moment, is shown in *Fig. 18*. It will be shown later that for soil in the dense state, a value of log ρ less than $-3{\cdot}0$ is not likely to be used. For the range of yield likely to occur in practice, the maximum negative moment will therefore be less than the maximum positive moment. In the case of loose soils, the question of tie-rod yield does not arise, since movements at the tie, much larger than those likely to occur in practice, are necessary before cantilever action operates.

Fig. 18



TIE-ROD YIELD NECESSARY BEFORE NEGATIVE MOMENT EXCEEDS MAXIMUM POSITIVE MOMENT

Effect of Vibrations

Tests were carried out in which the sides of the bin were subject to low-frequency high-amplitude vibrations at each stage of dredging until the gauge readings reached a maximum value.

The percentage-bending-moment/flexibility curves were similar to *Fig. 14* provided that the value of τ_{max} was calculated corresponding to active pressure with no wall-friction. The same result was obtained by allowing an outward yield of the sides of the bin. These tests confirmed the finding that the form of the percentage-bending-moment/flexibility curves was approximately independent of the value of the total moment acting on the pile. Where piling is likely to be subject to heavy vibrations, no wall friction should be assumed in design.

Effect of Water

Water in the soil may cause an increase or a decrease in the bending moment, depending on the levels of the water on each side of the pile.

Stationary or slowly flowing water will not change the relative density or compressibility of the soil. Since the shape and position of the moment/flexibility curves are independent of the value of the maximum moment on the pile, the curves may be used to calculate the acting bending moment, provided that the maximum Free Earth Support value τ_m is calculated to include the water-pressure. The results of tests made by Tschebotarioff with water present on both sides of the pile, have been plotted in *Fig. 14* as a percentage of the Free Earth value for the particular test. The results lie close to the general curves.

Influence of the Material of Construction and Shape of Section on the Behaviour of Field Piling

The flexibility of a field pile designed to withstand a particular bending moment at a certain stress, will depend upon the material of construction and the shape of the section. The simplest case is that of wood-piling of rectangular shape and uniform material.

1. *Wood-piling.*—

$$M = 2f \times \sqrt[3]{I^2} \text{ lb-inch per foot.}$$

If

$$\rho = \frac{H^4}{EI}, \quad \tau = \frac{M}{H^3},$$

then

$$\tau = \frac{2f}{\sqrt[3]{H(E\rho)^2}}$$

E and f are in lb. per square inch, I in inches ^{4} per foot, H in feet, and M in lb.-inches per foot.

Taking $E = 1{\cdot}5 \times 10^6$ lb. per square inch and $f = 2{,}000$ lb. per square inch,

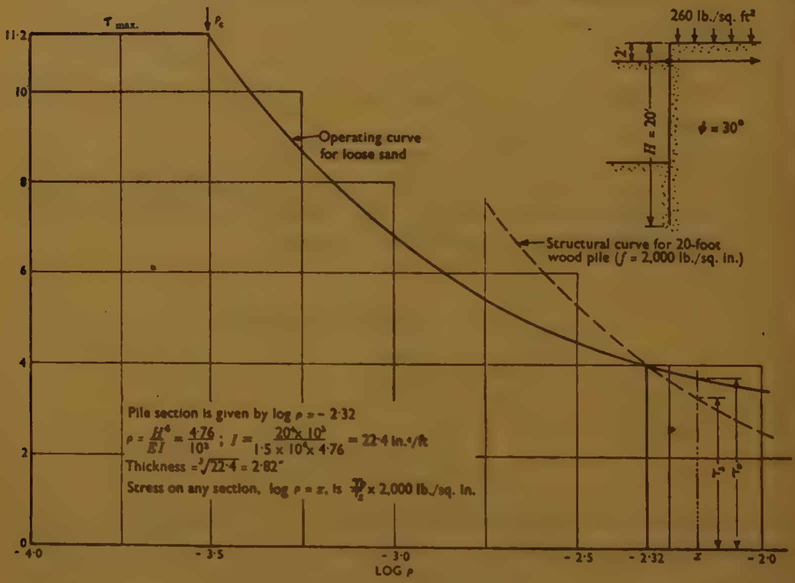
$$\tau = \frac{0{\cdot}30}{\sqrt[3]{H\rho^2}}$$

As an example, the relation between the flexibility number ρ of a wood pile of height $H = 20$ feet, and the moment τ it can withstand is plotted in *Figs 19* and denoted by the term "Structural Curve." In order to determine the pile section, the relation between the flexibility and the maximum bending moment operating on the structure must now be plotted on the same diagram. The maximum Free Earth Support bending moment is calculated, divided by the cube of the pile height, and a horizontal line is drawn on the diagram corresponding to this value τ_m. The critical flexibility ρ_c and moment reduction curve are interpolated from *Fig. 14* for the particular relative density of the subsoil, and are used to plot the "Operating Curve." The flexibility number corresponding to the intersection of the two curves gives the pile section at which the acting bending moment is equal to the maximum that the section is designed to withstand. The pile height H being known, the section modulus is determined.

2. *Steel-piling.*—The relation between I and y in this case depends upon the shape of section produced.

Empirical relations have been obtained from data supplied by the respective firms, and the following τ/ρ relations obtained for f=8 tons per square inch.

Figs 19



MOMENT/FLEXIBILITY CURVES

Dorman Long and Frodingham, $\tau = \dfrac{8{\cdot}15}{H^{0{\cdot}16}\rho^{0{\cdot}71} \times 10^{2}}$ *

Larssen, $\tau = \dfrac{0{\cdot}562}{H^{0{\cdot}54}\rho^{0{\cdot}615}}$

U.S.S. Carnegie-Illinois, $\tau = \dfrac{0{\cdot}638}{H^{0{\cdot}54}\rho^{0{\cdot}615}}$

3. *Reinforced concrete.*—The values of M and EI may be calculated from theory for a series of sections. Assuming equal areas of tension steel and compression steel, 1½ inch cover to centre of steel, and the

---

* Not applicable to box-piles.

experimental value of *EI* to be 1·26 times the theoretical value,[7] the following relations are obtained :—

$$\text{For design stresses, } c = 750 \text{ lb. per square inch}$$

$$\text{and } t = 18{,}000 \text{ lb. ,, ,, ,,}$$

$$\tau = \frac{0{\cdot}053}{H^{0\cdot2}\rho^{0\cdot70}}$$

$$\text{For design stresses, } c = 1{,}250 \text{ lb. per square inch}$$

$$t = 22{,}400 \text{ lb. ,, ,, ,,}$$

$$\tau = \frac{0{\cdot}062}{H^{0\cdot12}\rho^{0\cdot72}}$$

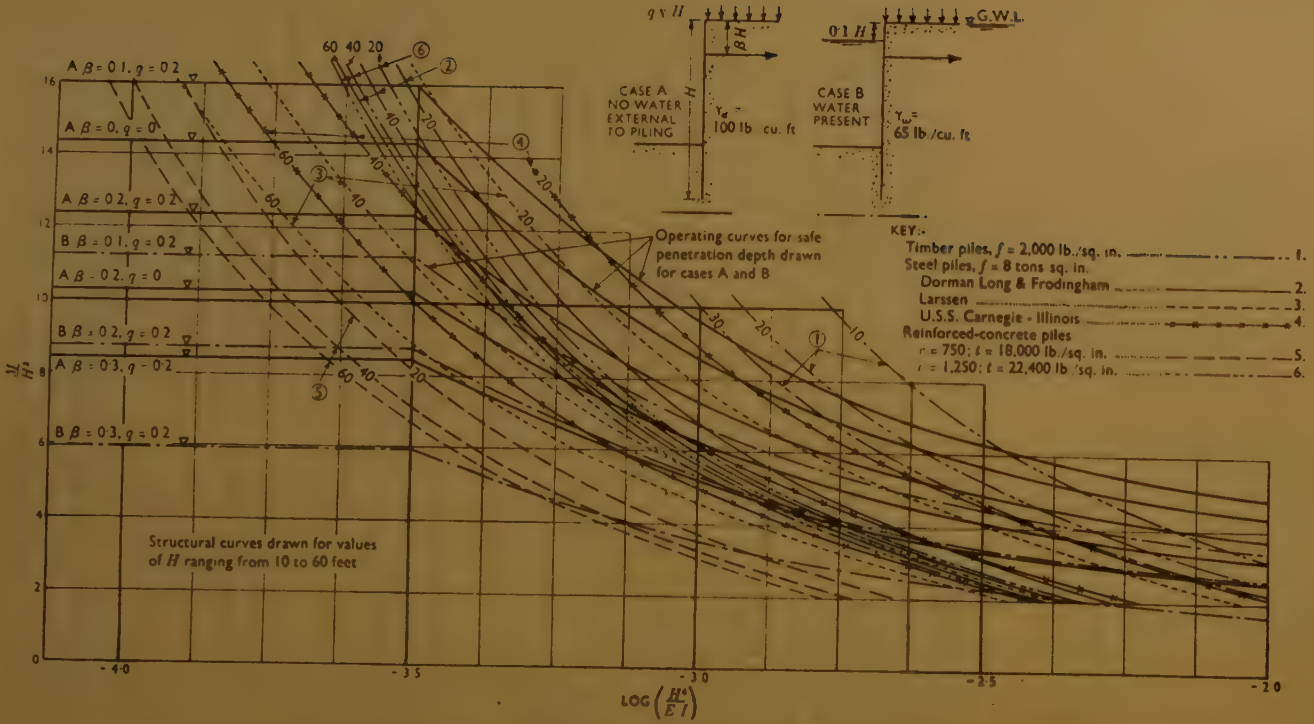
The structural curves corresponding to the above τ/ρ relation for wood, steel, and concrete piling, respectively, are plotted in *Figs 20* for three values of H in each case. On the same diagram are plotted the operating curves corresponding to a number of conditions of tie-rod level, surcharge, and water-level, determined in each case from the maximum Free Earth Support moment τ_m, and the reduction curves in *Fig. 14.* All the operating curves are for loose sand.

It is seen that structural curves for the concrete sections designed to withstand a maximum concrete stress equal to 750 lb. per square inch, cut the operating curves below the critical flexibility, so that for this soil, these structures operate after the manner of Free Earth Support. Wood-piling, in general, operates after Fixed Earth Support. Steel-piling and concrete which is designed to higher stresses take an intermediate position. The degree of fixity of the optimum pile section depends, however, on the values of β and q, on the presence of water, on the height of the piling, and on the exact shape of the section, as well as on the structural material and soil condition.

A more general picture is shown in *Figs 21* where mean structural curves for steel and concrete are drawn, and the influence of soil relative density is illustrated by the operating curves for extreme and intermediate field conditions. This diagram shows that for the type of structure shown in (B), *Figs 21,* with subsoil of average relative density, the optimum design leads to an operating bending moment on steel and high-strength-concrete sections of 44 per cent of the maximum Free Earth value. Such a structure built in low-strength concrete should be designed to withstand 82 per cent of the same maximum moment. For the extreme conditions of loose soil and water which may recede to the dredge level, both types of piling should be designed to withstand the full Free Earth moment, whilst for dense soil and water level with the coping and low tie-rods, even the low-strength-concrete section may be designed at complete fixity.

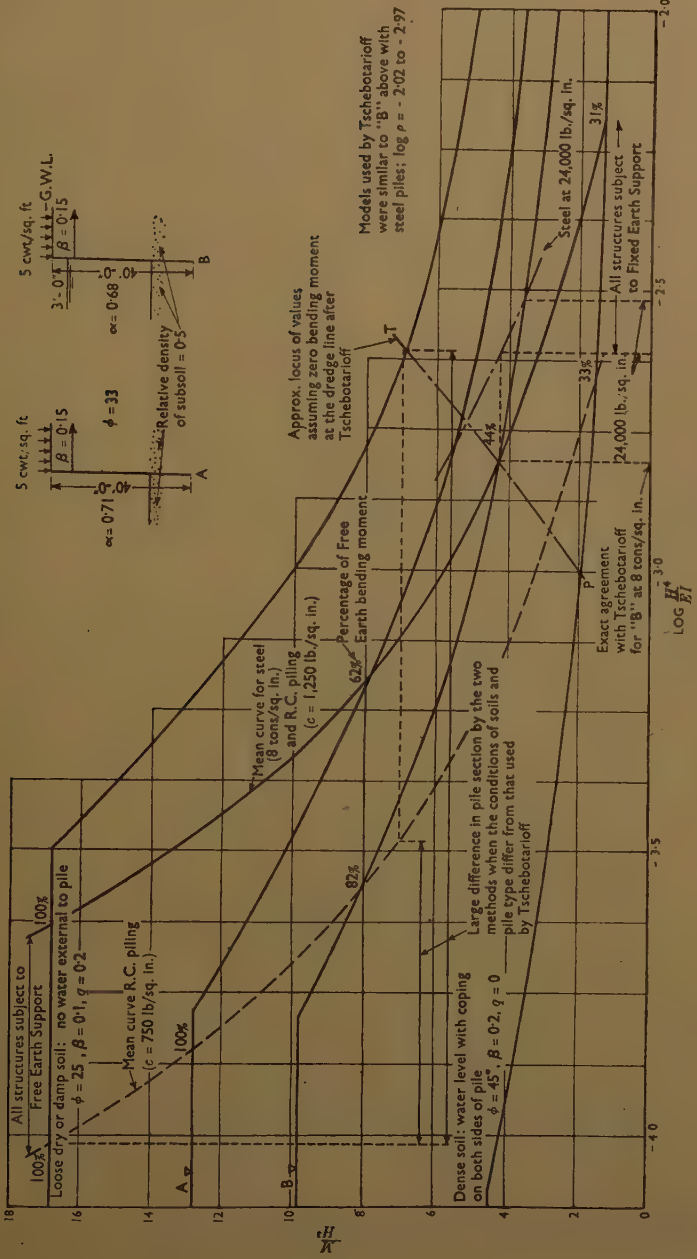
The diagram also shows the position of Tschebotarioff's experiments in the general range of possible conditions. His models simulated type

Figs 20



Proposed Form of Design Chart for Optimum Size and Type of Piling in Bending.
Curves for Anchor Yield. Loose Soil, $\phi = 30°$

Figs 21



GENERAL RANGE OF FIELD CONDITIONS

(B) and were made of steel. His design procedure gave an operating bending moment just under one-half of the Free Earth Support value, and the exact values for the four conditions taken in *Figs 21* have been calculated and plotted to form the line PT. Exact agreement is seen between Tschebotarioff's value and the present proposals for the particular case investigated. However, for the extreme condition of a reinforced-concrete pile designed to 750-lb.-per-square-inch concrete stress, driven in loose soil, and with no water external to the piling, the bending moment calculated by Tschebotarioff would be given by point T, that is to say, $\tau=7{\cdot}1$. At this value, the design flexibility in low-strength concrete would be given approximately by $\log \rho=-3{\cdot}5$. With this section, the actual operating moment would be given by $\tau=16{\cdot}8$, that is to say, more than twice the design value.

The graphs, *Figs 20* and *21*, show that conditions of Free Earth Support or Fixed Earth Support cannot be assumed to act irrespective of tie-rod level, surcharge value, soil properties, and structural material. In addition, it is of interest to recall that the Danish Rules were formed for concrete piling from the observation of the stability of wooden piling.[8] The graphs show how this procedure has led to higher stresses in the concrete and reinforcements than had been supposed.

Tie-Rod Loads

The load varies from the Free Earth Support value as follows :—

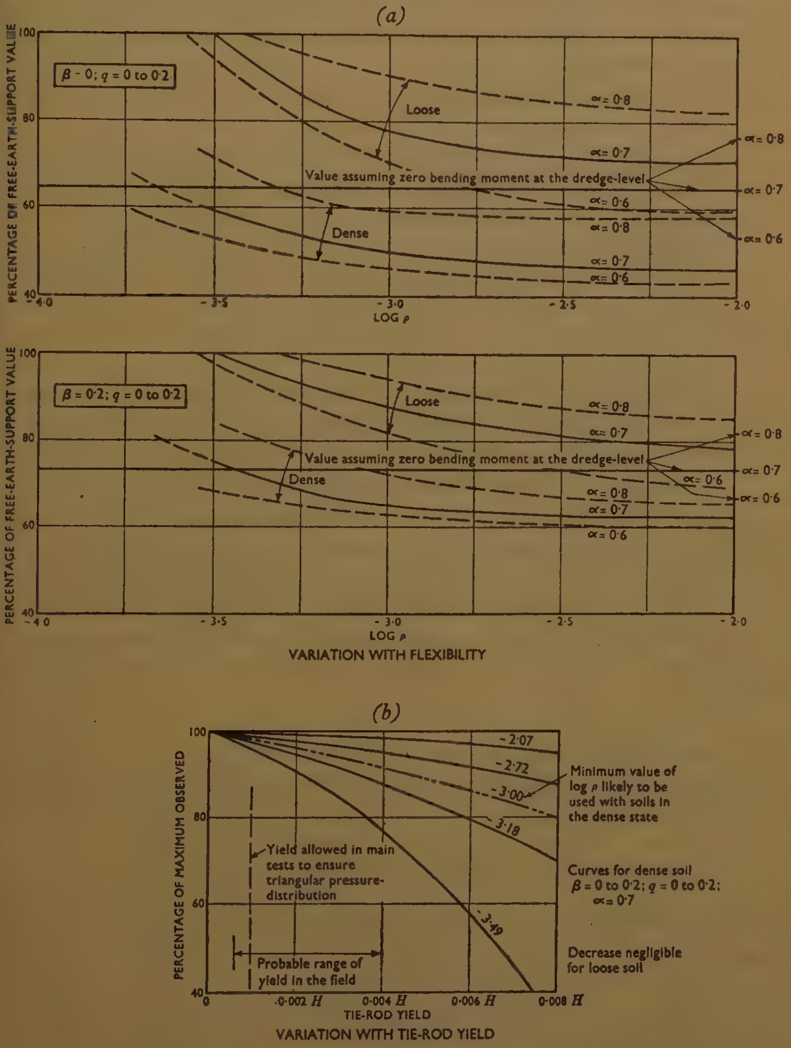
1. Decrease with increase in flexibility due to decrease in effective span. The curves are shown in *Figs 22* (*a*).
2. Decrease with tie-rod yield with dense soils. The curves in *Fig. 22* (*b*) show that this may generally be neglected.
3. Increase due to differential yield of the tie-rods. The load on any one tie-rod may be increased by 50 per cent of the mean load due to a settlement at the anchor plate of the adjacent rod by an amount equal to the natural deflexion of the waling. In the larger-model experiments, the increase ranged from 20 to 50 per cent, even though great care was taken in setting up.
4. Increase due to surcharge load transmitted directly to the tie-rods with soil settlement.

Assuming that provision is made to prevent the occurrence of (4), the maximum values of tie-rod load operate when differential yield acts with a concrete pile designed to low stresses. It has been seen that the economical design in most cases will lead to the use of a section of higher flexibility so that tie-rod load due to (1) and (3) above will not be likely to exceed the Free Earth value.

Depth of Penetration

This must be sufficient to prevent both passive failure and movements at the toe large enough to cause settlement of the retained soil. In addition, it has been customary to drive deeper to ensure "fixity" support

Figs 22



Variation in Tie-Rod Load with Pile Flexibility and Tie-Rod Yield

in accordance with the assumption that complete fixity occurs at a certain depth below the dredge line, independent of the pile section and soil state. Driving deeper does tend to increase fixity conditions, but if the soil is loose and the section stiff, a very considerable increase in the length of the pile is necessary to produce complete fixity. On the other hand, if the soil is dense, and the section flexible, fixity will exist at very low values of the dredge level and nothing could be gained by deep driving.

Diagrams of the stages leading to complete failure for stiff and flexible piling are given in *Figs 23*, and are based on direct measurements. These show the following results :—

1. Both stiff and flexible piling is in a state of Free Earth Support at the instant of failure.
2. A considerable shear force acts at the toe of the pile at this stage, owing principally to the vertical component of the active pressure.
3. During dredging, movements at the toe of stiff piling in loose soil became appreciable only after the Coulomb passive-pressure distribution with no wall friction had been fully mobilized. No forward movements at the toe occurred with stiff piling in dense soil or flexible piling in loose soil until very low values of the dredge level were reached.
4. For stiff piling, dredged to levels consistent with safe design in the past, the Coulomb ($\delta=0$) passive-pressure distribution is partly mobilized as assumed by Krey. The resultant of this pressure and the toe shear force acts approximately at $\frac{1}{3}D$ from the toe.

In view of (1) all piling will have the same ultimate factor of safety if driven to adequate depth on the assumption of Free Earth Support. The curves in *Figs 23* have been calculated using a factor of safety equal to 1·5 on the Coulomb ($\delta=0$) passive load, with the inclusion of the shear force at the toe. This gives a total factor of safety of approximately 3 against complete slip, owing to the fact that full wall-friction is mobilized at failure.

Maximum Outward Deflexion due to Bending

The deflexion of a simply supported beam subject to triangular loading is given by :—

$$d \propto \frac{(\xi H)^5}{EI} = \xi^5 H \rho, \text{ where } \xi H \text{ is the free span.}$$

The maximum moment $M \propto (\xi H)^3$

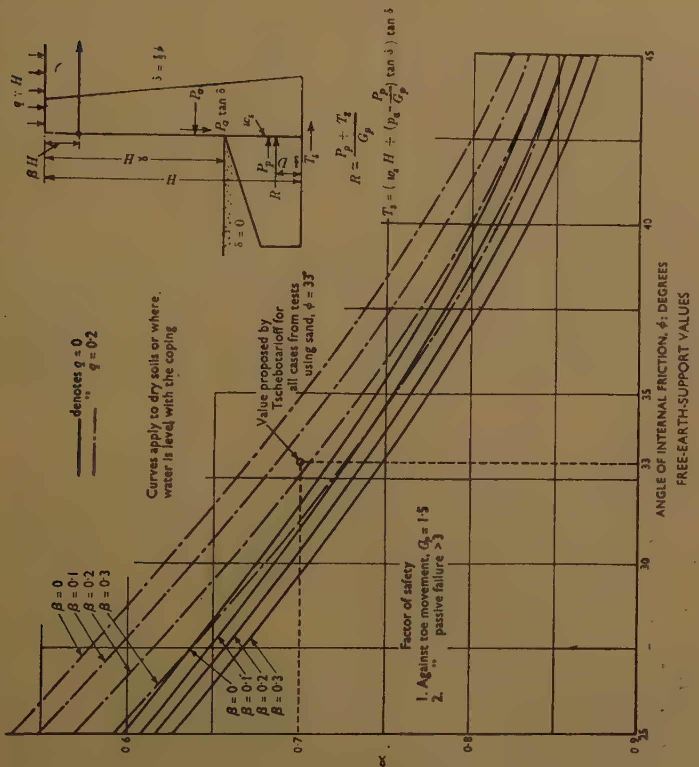
that is to say, $\tau \propto \xi^3$

For steel-piling $\tau \propto \frac{1}{H^{0\cdot 16} \rho^{0\cdot 71}}$

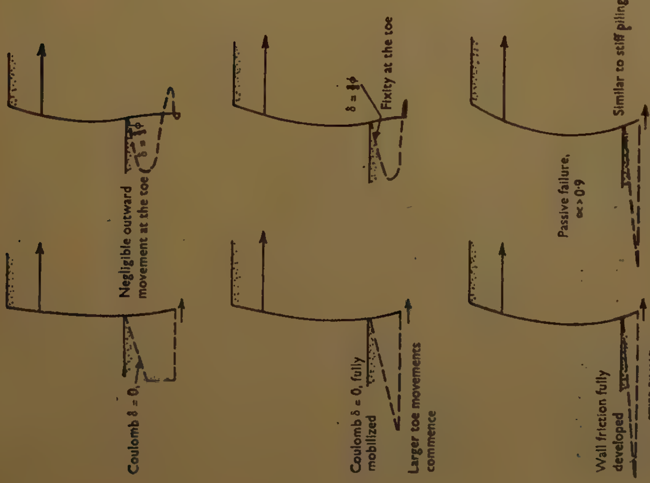
Therefore, $\frac{d}{H} \propto \tau^{5/3} \rho \propto \frac{\rho}{H^{0\cdot 27} \rho^{1\cdot 18}} = \frac{1}{H^{0\cdot 27} \rho^{0\cdot 18}}$

or approximately, $\frac{d}{H} \propto \frac{1}{\sqrt[4]{H} \times \sqrt[5]{\rho}}$

Figs 23



PENETRATION DEPTHS



Coulomb δ = 0,
Negligible outward movement at the toe
δ = ⅔ φ
Coulomb δ = 0, fully mobilized
Larger toe movements commence
δ = ⅔ φ
Fixity at the toe
Wall friction fully developed
STIFF PILING
Passive failure, α > 0·9
FLEXIBLE PILING
Similar to stiff piling

STAGES IN PASSIVE PRESSURE DISTRIBUTION AND DEFLEXION LEADING TO PRASSIVE FAILURE

That is to say, the deflexion decreases slightly with increase in flexibility. Further, the point of zero bending moment will move outward with flexibility increase, and these two factors combine to cause a slight increase in deflexion with flexibility. The larger flexibilities, however, only occur in design where the soil conditions are favourable and lower tie-rod positions are used. For these reasons, the maximum deflexions will occur with piling of intermediate stiffness.

The mean structural curve for steel at 8 tons per square inch stress, was used to calculate the maximum deflexions of field piles. The bending-moment distribution corresponding to the point of intersection of the operating and structural curve for each series of tests was integrated. The maximum deflexion due to bending so obtained was approximately $0{\cdot}004H$ for $\beta=0{\cdot}1$ and $D_r=0{\cdot}5$, and did not exceed $0{\cdot}006H$ for $\beta=0$ and $D_r=0$.

Bulkheads Anchored to Non-yielding Platforms

All the tests made with non-yielding anchors with stiff piling, showed close agreement with Stroyer's results in the value of the bending moment. This relief in bending moment is in addition to that due to flexure, so that Stroyer's reduction factor may be applied to the value of τ_m before drawing the $\tau/\log \rho$ curve for design. The reduction factor is $\frac{2K_a}{1-K_a{}^2}$ where K_a is the Coulomb active-pressure coefficient with no wall-friction.

The concentration of pressure at the tie-rod level caused an increase in the tie-rod load which increased with tie-rod depth. For the type of structure under consideration, $\beta=q=0$, and the load increase in this case was of the order of 10 per cent. No increase due to differential yield can occur.

The decrease in overturning moment with soil-arching was calculated to allow a reduction in the pile-length by approximately 3 per cent.

Examples of Design Method and Comparison with Fixed Earth Support Analysis

A graphical analysis of a structure is given in *Figs 24*.

The lateral active pressures are calculated taking full wall-friction, and the passive pressures are taken as $\frac{1}{1{\cdot}5}$ times the value with no wall-friction. It is clear that the penetration will exceed 13 feet, and the length of the piling below this is unknown and denoted by x. The resultant pressure diagram is divided into a number of small areas and the point loads equivalent to each area are calculated. The value of the toe shear force is determined in terms of the unknown x; and the value of x—and hence the sheeting height H—is found by taking moments of all forces about the anchor-level. For the reasons already given, the moment of the toe shear

Figs 24

ACTIVE EARTH PRESSURE COEFFICIENT: $K_a = \dfrac{\cos^2\phi}{\left[1 + \sqrt{\dfrac{\sin(\phi+\delta)\sin\phi}{\cos\delta}}\right]^2}$

PASSIVE EARTH PRESSURE COEFFICIENT: $K_p = \dfrac{1}{1{\cdot}5}\tan^2(45 + \tfrac{1}{2}\phi)$

$\delta = \tfrac{2}{3}\phi$

$\phi°$	K_a	K_p	γ_{air}	γ_{water}
35	0·225	—	105	73·7
31	0·270	—	95	60·3
36	0·215	2·6	105	67·1
42	0·165	3·4	110	69·2

RESULTANT EARTH LOAD ON PILING

$$\Sigma P_a - \Sigma P_p = 16{\cdot}44 - [1{\cdot}37 + 0{\cdot}557x + 0{\cdot}05x^2]$$

SHEAR FORCE AT THE TOE

$$T_s = \frac{1}{1{\cdot}5}\tan\delta[\Sigma(P_a - P_p)\tan\delta + w_s H]$$

Taking $w_s \doteqdot 50$ lb./sq. ft. $H \doteqdot 60$ ft.

$$T_s = 3{\cdot}25 - 0{\cdot}10x - 0{\cdot}009x^2$$

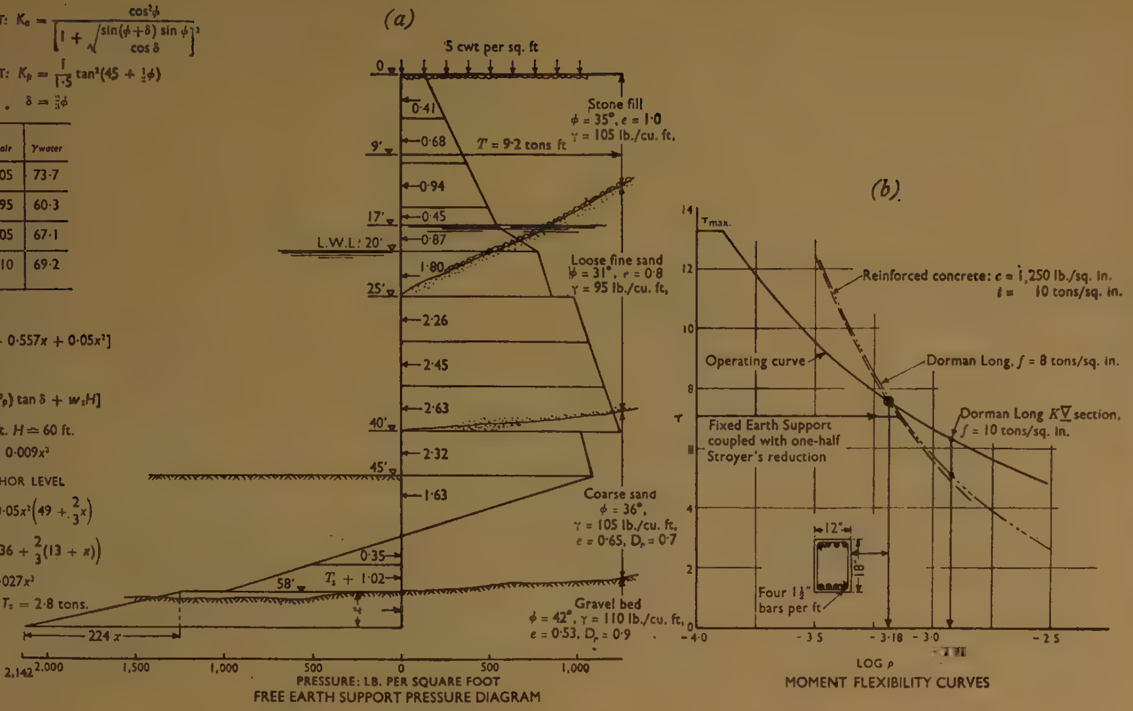
TAKING MOMENTS ABOUT THE ANCHOR LEVEL

$$285{\cdot}4 = 0{\cdot}557x\left(49 + \frac{x}{2}\right) + 0{\cdot}05x^2\left(49 + \frac{2}{3}x\right)$$

$$+ (3{\cdot}25 - 0{\cdot}10x - 0{\cdot}009x^2)\left(36 + \frac{2}{3}(13 + x)\right)$$

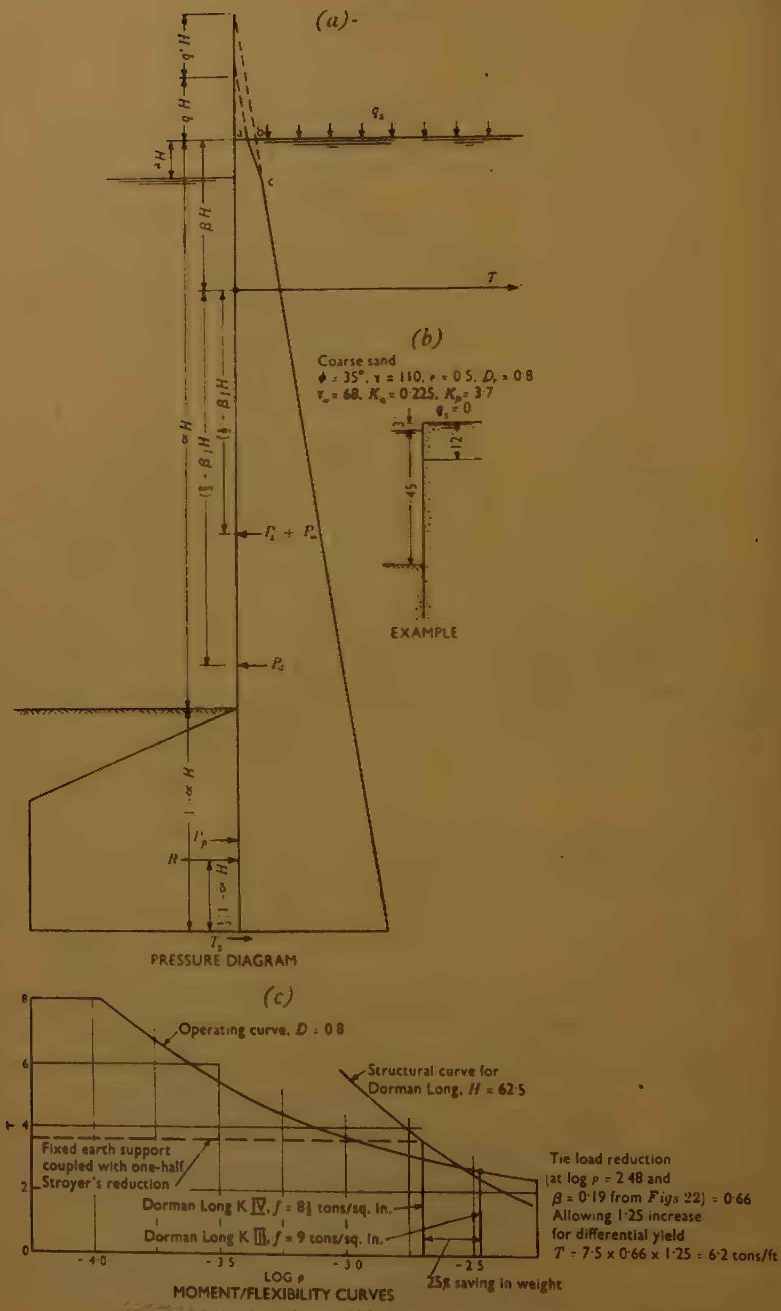
$$140 = 24{\cdot}9x + 2{\cdot}26x^2 + 0{\cdot}027x^3$$

$x = 4{\cdot}0$ ft.; $H = 62$ ft.; $T_s = 2{\cdot}8$ tons.



EXAMPLE OF PILING SUBJECT TO LARGE VALUE OF τ_m

Figs 25


(a)
q'H
qH
q_s
a
b
c
rH
βH
T
αH
(1 - β₁)H
(½ - β₁)H
$P_s + P_w$
P_a
1 - αH
P'_p
R
$\frac{1}{3}$(1 - α)H
T_s
PRESSURE DIAGRAM
(b)
Coarse sand
φ = 35°, γ = 110, e = 0·5, D_r = 0·8
γ_w = 68, K_a = 0·225, K_p = 3·7
q_s = 0
3
12
45
EXAMPLE
(c)
Operating curve, D = 0·8
Structural curve for Dorman Long, H = 62·5
Fixed earth support coupled with one-half Stroyer's reduction
Dorman Long K IV, f = 8½ tons/sq. in.
Dorman Long K III, f = 9 tons/sq. in.
Tie load reduction (at log ρ = 2·48 and β = 0·19 from Figs 22) = 0·66
Allowing 1·25 increase for differential yield
T = 7·5 x 0·66 x 1·25 = 6·2 tons/ft
25% saving in weight
τ
8
6
4
2
0
- 4·0
- 3·5
- 3·0
- 2·5
LOG ρ
MOMENT/FLEXIBILITY CURVES

EXAMPLE OF PILING SUBJECT TO SMALL VALUE OF τ_m

ANALYTICAL METHOD SUITABLE FOR UNIFORM SOIL (SEE *Figs 25*)

Approximations

(1) Pressure area abc added to water pressure.

(2) Resultant of P_p and T_s assumed to act at $\frac{1}{3}$ penetration depth from toe.

Penetration Depth

$$P_a = \frac{K_a\gamma_w H^2}{2},\ P_s = qK_a\gamma_w H^2;\ P_w = q'K_a\gamma_w H^2;\ q = \frac{q_s}{\gamma_w H};\ q' = \frac{w\theta}{\gamma_w K_a};$$

$$P_p = \frac{K_p\gamma_w}{2G_p}(1-\alpha)^2H^2$$

$$T_s = \frac{\tan\delta}{Gp}\left[(P_a + P_s - P_p)\tan\delta + W_sH\right];\ R = P_p + T_s$$

Taking moments about the tie-rod level

$$P_a(\tfrac{2}{3} - \beta)H + (P_w + P_s)[\tfrac{1}{2} - \beta]H = R\left[1 - \beta - \frac{(1-\alpha)}{3}\right]H$$

Substituting for P_a, P_w, P_s, R, P_p and T_s, and dividing by $K_a\gamma_w H^3$

$$\tfrac{1}{2}[\tfrac{2}{3} - \beta] + (q + q')[\tfrac{1}{2} - \beta] = \frac{(2 + \alpha - 3\beta)}{3G_p}\left[\frac{(1-\alpha)^2}{2}\frac{K_p}{K_a}\left(1 - \frac{\tan^2\delta}{G_p}\right) + (q + \tfrac{1}{2})\tan^2\delta + \frac{W_s\tan\delta}{K_a\gamma_w H}\right] \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Assume value of $\alpha \simeq 0{\cdot}75$ for this example at this stage

$H \simeq \frac{45'}{0{\cdot}75} = 60'$; $\theta \simeq \frac{3}{60} = 0{\cdot}05$; $\beta \simeq \frac{12}{60} = 0{\cdot}2$; $q = 0$; $q' \simeq \frac{62{\cdot}5 \times 0{\cdot}05}{68 \times 0{\cdot}225} = 0{\cdot}2$

$W_s \simeq 24$ lb./ft.^{2}; $\tan\delta \simeq 0{\cdot}43$; $G_p = 1{\cdot}5$

Equation (1) gives $\alpha = 0{\cdot}72$; hence $H = 62{\cdot}5'$; $\beta = 0{\cdot}192$; $q' = 0{\cdot}197$

Tie-rod Load

Taking moments about the point of action of force R

$$T\left[\frac{2+\alpha}{3} - \beta\right]H = \frac{K_a\gamma_w H^3}{2}\left[\frac{1}{3} - \frac{(1-\alpha)}{3}\right] + (q + q')K_a\gamma_w H^3\left(\frac{1}{2} - \frac{(1-\alpha)}{3}\right)$$

$$\left(\frac{T}{H^2}\right) = \frac{K_a\gamma_w}{2}\left[\frac{\alpha + (q + q')(1 + 2\alpha)}{2 + \alpha - 3\beta}\right] \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$\frac{T}{H^2} = 4{\cdot}28 \text{ lb./ft.}^3;\ T = \frac{4{\cdot}28 \times 62{\cdot}5^2}{2240} = 7{\cdot}5 \text{ tons/ft.}$$

Bending Moment

At depth ηH below the surface $\beta < \eta < \alpha$

$$\frac{M}{H^3} = \left[\frac{T}{H^2}(\eta - \beta) - \frac{K_a\gamma_w}{6}(\eta^3 + 3\eta^2(q + q')\right] \times 12 \text{ lb. in./ft.} \quad . \quad . \quad . \quad (3)$$

Max. value τ_m occurs at $\bar{\eta}$ when $\dfrac{\delta\left(\frac{M}{H^3}\right)}{d\eta} = 0$

$$\bar{\eta} = \sqrt{(q + q')^2 + \frac{2}{K_a\gamma_w}\left(\frac{T}{H^2}\right)} - (q + q')$$

$$\bar{\eta} = 0{\cdot}574;\ \tau_m = 8{\cdot}0$$

force is calculated as if it acted at the centre of pressure of the passive-earth-pressure diagram.

The maximum bending moment resulting from the system of forces may then be determined graphically in the normal way. The value obtained (117 tons-feet) is converted to lb.-inch-per-foot units and divided by the cube of the height (in feet) of the sheeting. The result is the value

$$\text{of } \tau_m \left(= \frac{117 \times 2240 \times 12}{62^3} = 13{\cdot}2 \text{ lb.-inch/ft}^4 \right).$$

The moment operating curve is drawn by applying the percentage-reduction values from *Fig. 14*, interpolated for $D_r = 0{\cdot}7$ and this is shown in *Fig. 24* (*b*). The structural curve for a 62-foot reinforced-concrete or steel pile is added and the intersection point gives the value of $\rho = \frac{H^4}{EI}$ for the section and hence the value of *EI*.

The Free Earth Support tie-rod load may be reduced by a factor 0·7 from *Figs 22*, but should be increased by a factor of 1·25 to allow for differential yield.

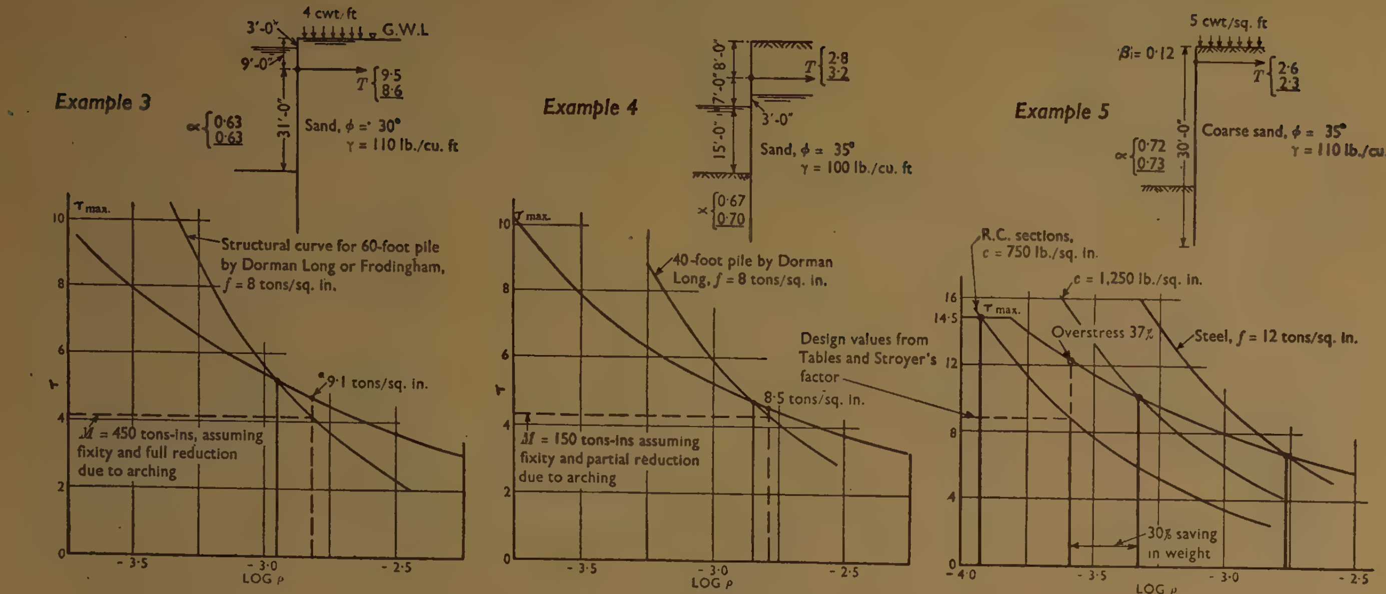
A second example is given in *Figs* 25 where the soil, water-level, anchor-level, and surcharge conditions result in a lower value of τ_m. The uniform soil is taken to illustrate the direct calculation of the Free Earth values.

The moment/flexibility curves for three existing structures are given in *Figs* 26.

The proposed design procedure will generally lead to economy compared with the use of the Free Earth Support method coupled with Stroyer's reduction factor, or with the Fixed Earth Support analysis with no allowance due to arching. However, it is of interest to compare the design quantities in the first two examples with the Fixed Earth Support analysis coupled with either one-half of Stroyer's reduction or a general 25-per-cent reduction. For these first two examples, full wall-friction was allowed on the active-pressure side and one-half full friction on the passive side, in the construction of the Fixed Earth Support diagram. The penetration depth determined was multiplied by a factor of 1·2, and the tie-rod load by 1·1. For examples 3 [9] and 4,[10] the values taken are those given by the original designers. All values are shown in Table 4.

The comparison shows that the Fixed Earth Support analysis coupled with a further reduction factor has estimated the optimum pile section very closely, but that this method may have led to the use of larger penetration depths than are really necessary to maintain fixity. The comparisons depend however on the relative density of the soil, which has not been previously measured. In general, the proposed method gives a conservative value for the pile section where the soil and surcharge conditions are unfavourable, that is to say, when τ_m approaches a value of 16, while economy results for favourable conditions where τ_m approaches 8.

Figs 26



COMPARISONS WITH PREVIOUS DESIGNS

Operating curves are drawn assuming relative density of soil=0·5

Anchor loads and penetration compared thus : $\alpha \begin{cases} 0{\cdot}63\text{—previous design} \\ \underline{0{\cdot}63}\text{—recalculated} \end{cases}$

TABLE 4

Example	Method*	Penetration: feet	Max. mom. per unit height: lb. in./ft.^{4}	Tie-rod load: tons/ft.	Remarks
1.	P	17	7·5	8·0	Unfavourable condition in bending due to loose sand at centre of piling, surcharge and water pressure. New method gives conservative moment and tie load. Good subsoil—less penetration.
	F–½S	24	7·1	7·8	
	F–R		6·8		
2.	P	17·5	2·7	6·2	Good condition. Medium value of ϕ, all soil at reduced density in water, no surcharge and lower tie rods than in Ex. 1. New method gives all-round economy.
	F–½S	21	3·6	6·4	
	F–R		3·6		
3.	P	22·5	5·2	8·6	Medium conditions. Ex. 3 has surcharge and $\phi=30°$ but soil fully submerged. Ex. 4 has no surcharge and $\phi=35°$ but soil only partially submerged. Close agreement between methods where only half arching reduction was used.
	O.D.	22·5	4·2	9·5	
	F–½S		5·1		
	F–R		4·8		
4.	P	12·8	4·7	3·2	
	F-½S(O.D.)	14·5	4·3	2·8	
	F–R		4·2		
5.	P	8·5	10	2·3	Tidal conditions. Use of higher-strength-concrete or steel section may lead to economy.
	F$_r$–S	8·5	8·5	2·6	

* Methods.
P denotes proposed method.
F–½S „ Fixed Earth Support reduced by ½ Stroyer's factor. ($\delta=\frac{2}{3}\phi$ active, $\delta=\frac{1}{2}\phi$ passive.)
F–R „ Fixed Earth Support reduced by 25 per cent.
O.D. „ original designers' values.
F$_r$–S „ Free Earth Support reduced by full value of Stroyer's factor.

Example No. 5[11] is of a reinforced-concrete pile previously designed to low stresses, using Free Earth Support analysis $\delta=0$ and Stroyer's reduction value. It is seen that the section would be overstressed by 37 per cent and that by designing to the higher concrete stress by the new method, a 30-per-cent saving in weight of concrete is obtained. In addition, the use of higher-strength-steel piling may prove economical for cases of this type where the water may recede to the dredge level.

Factor of Safety in Bending

The use of a stress exceeding 8 tons per square inch for steel piling will generally allow further moment reduction and consequently larger economy in material than would occur with a structure subject to a constant moment. In addition, if the yield point were reached, the steel could only elongate at the point of maximum stress if the pile rotated further into the subsoil, and this would generally lead to a further decrease in moment. Larger deflexions would occur but the structure would remain in stable equilibrium. For these reasons, the use of steel stresses of 10 to 12 tons per square inch is worthy of consideration where a thorough survey of the subsoil has been made and the maximum surcharge and water-pressures can be estimated accurately.

SUMMARY

While the experimental results showed agreement with the work of Stroyer, Browzin, and Tschebotarioff, for the same model conditions, the maximum bending moment was found to depend upon the flexibility of the piling and the relative density and compressibility of the soil below the dredge level.

It is proposed that the Free Earth Support analysis be used as a basis for design, taking the following forces,

1. Active earth pressure : Coulomb value, $\delta = \frac{2}{3}\phi$.
2. Passive earth pressure : $\frac{1}{1 \cdot 5} \times$ Coulomb value $\delta = 0$.
3. A shear force at the toe of the pile.

The maximum bending moment τ_m may then be used to draw the moment/flexibility curve from the data in *Fig. 14*. Stroyer's reduction factor may be applied to the value of τ_m for driven and dredged piling subject to no anchor yield.

The anchor load is decreased on flexible piling and is increased by differential yield or by pressure concentration at the anchor level. The resultant load is generally close to the Free Earth Support value.

ACKNOWLEDGEMENTS

The work was carried out in the Soil Mechanics Laboratory of the Engineering Department, University College, Dundee, St. Andrews University.

The Author expresses his thanks to Professor W. T. Marshall, Ph.D., B.Sc. (Eng.), A.M.I.C.E., for his advice and for the facilities provided for this work. Professor G. P. Tschebotarioff kindly supplied the Author with a complete record of all his work on sheet-piling.

The Paper is accompanied by three photographs and twenty-three sheets of diagrams, from which the Tables, the half-tone page plates, and the Figures in the text have been prepared.

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Paper No. 5838

"Tests on Full-Scale Portal Frames"

by

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(Ordered by the Council to be published with written discussion.)†

SYNOPSIS

The Paper describes tests to collapse on rectangular welded portal frames subjected to vertical and horizontal loads. In each case the portal was made up of 8-inch-by-4-inch joist sections throughout and had a span of 16 feet and a height of 8 feet. The portals were tested in pairs at 12-foot spacing.

Strain measurements were made at 150 positions in the elastic range and deflexions were measured throughout.

The first two portals tested had pinned bases. Each was subjected to a central vertical load V and a horizontal load H, of the same magnitude, at beam-level. These portals collapsed when $V = H = 5{\cdot}75$ tons which was in close agreement with the collapse load, 5·65 tons, calculated according to the simple plastic theory.

The next two portals had fixed bases, a single welding run with a No. 8 gauge electrode being made around the profile of the joist section at each base, with two extra runs added on the outside of each flange. The horizontal load applied was twice the vertical, and the calculated collapse load was $H = 2V = 11{\cdot}3$ tons. Though large permanent deflexions were produced at this load, collapse did not occur and the portals supported the full available load, $H = 2V = 13{\cdot}11$ tons, for 3 days without collapsing completely.

The last two portals were exactly the same as the previous set except that the extra two runs of welding were not made on the outside of the flanges at the bases of the stanchions. These portals supported more than the calculated load but collapse occurred owing to failure of the welds at the stanchion bases when $H = 2V = 12{\cdot}64$ tons.

It is considered that the fixed-base portals carried more than the calculated collapse load because of strain hardening at the feet of the stanchions.

INTRODUCTION

ADVANCES are more difficult to make in civil and structural than in other branches of engineering because a straightforward empirical approach is not possible. The monumental nature of most civil engineering work means that a test to destruction of an actual structure is often impracticable and almost always prohibitively expensive. This handicap can, to some extent, be minimized by developing a new theory with the help of experiments on small-scale structures and then carrying out a few carefully selected tests on the full scale to check and refine the theory. This method has been followed in the investigation into the behaviour of steel frames in the plastic range[1] which has been carried out in recent years. The stage has now been reached when the theory can be applied to the design of single-storey shed buildings.[2] The rectangular portal—the simplest form of this type of structure—has been studied in some detail and a satisfactory procedure evolved for proportioning the members so that the structure will collapse under a permissible factored load.[3] The basic assumptions underlying the method have been fully confirmed by a comprehensive series of tests on miniature portal frames.[4]

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† Correspondence on this Paper should be received at the Institution by the 1st April, 1952, and will be published in Part I of the Proceedings. Contributions should be limited to about 600 words.—SEC. I.C.E.

Tests have now been made on full-scale rectangular portals, two with pinned, and four with fixed bases, so that their behaviour under severe conditions of loading, which carry them out of the elastic into the plastic range, can be compared with that of the small-scale structures on which the simple theory has been based.

General Description of Experimental Site

The experiments were carried out at the structural testing ground at the Abington Research Station of the British Welding Research Association. The general arrangement of the test frames and loading rig is shown in *Figs 1* and *2* for frames with pinned base and with fixed base stanchions respectively. To obtain the desired conditions at the bases, the stanchions of the former frame were mounted on pins running in ball races, and for the latter, by securing them to a heavy compound girder. In both cases the portals, of 8-inch-by-4-inch joist section throughout, had a span of 16 feet and a stanchion height of 8 feet, and were tested in pairs at 12-foot spacing, so that a stable framework could be obtained by the use of wire bracing in the side and top panels. Vertical load was applied by means of a pair of steel tanks, A1 and A2 (*Fig. 1*) suspended from the joist C, supported in turn on the portal frames at the mid-points of the beams. Horizontal load was applied by a second pair of tanks, A3 and A4, each suspended from a pair of steel cables passing over a large pulley in the frame G, and attached to the portal frame through a heavy lug welded on at the top of the stanchion and on the centre-line of the beam.

The whole test rig was erected on a reinforced-concrete foundation of the raft type, consisting of a 9-inch slab over 20-inch-by-18-inch beams, the latter spanning in both directions and intersecting at 16-foot centres in one direction and 12-foot centres in the other, to coincide with the span and spacing respectively of the portal frames.

Load was provided in the form of lengths of caterpillar track links, and

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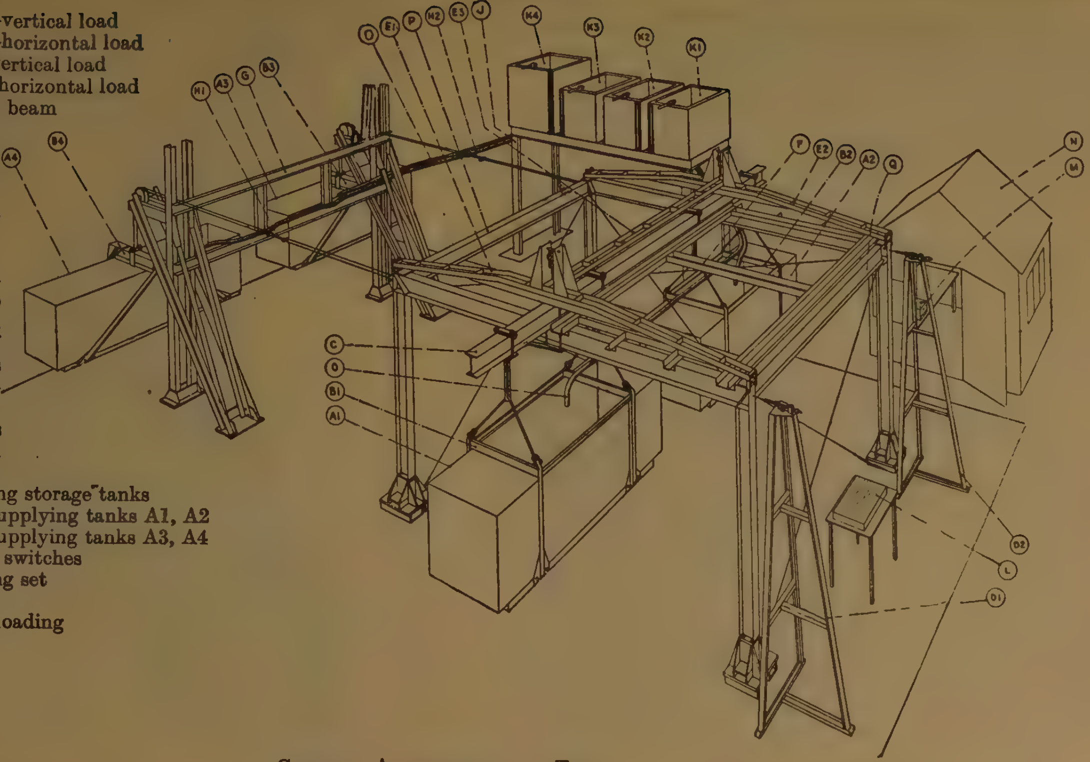
[1] The references are given on p. 94.

Fig. 1

KEY	
A1, A2	Loading tanks—vertical load
A3, A4	,, ,, —horizontal load
B1, B2	Tank cradles—vertical load
B3, B4	,, ,, —horizontal load
C	Vertical loading beam
D1, D2	Stands supporting horizontal deflexion gauges
E1, E2	Frames carrying vertical deflexion gauges
F	Frame providing lateral support to top flanges on windward half of portal beams
G	Horizontal loading frame
H1, H2	Tension links in horizontal loading cables
J	Frame supporting storage tanks
K1, K2	Storage tanks supplying tanks A1, A2
K3, K4	Storage tanks supplying tanks A3, A4
L	Maihak selector switches
M	Maihak recording set
N	Hut
O	Hose for water loading
P, Q	Tie beams



General Arrangement of Test

by water run in from four storage tanks mounted on a frame J. Each tank, holding about 1 ton of water, was connected to a particular loading tank by a hose line and the increment of load was controlled by a valve in the bottom of the storage tank, and recorded on a graduated tube.

The frames E, simply supported at the tops of the stanchions, were provided to hold a simple gauge for recording the central deflexion of the beam. Another pair of frames D, secured to the concrete foundation, supported similar gauges for recording the horizontal deflexion. In addition to these records, strains were measured at three sections on the stanchions and six sections on the beams using Maihak acoustic strain gauges, all of which were connected to a bank of selector switches at L and in turn to a recording set M in the hut at N.

DESIGN AND CONSTRUCTION OF FRAMES AND TESTING RIG

Design Data for Details

The 8-inch-by-4-inch joist section and the overall dimensions of a 16-foot span and an 8-foot stanchion height for these portals were chosen to conform with those of the three-storey experimental frame of bolted construction, erected in 1930 at the Building Research Station for the investigations of the Steel Structures Research Committee.[5, 6] This has the advantage that as the work develops, comparison can readily be made with frames of semi-rigid construction.

To determine the loads and reactions for which provision would have to be made, consideration was given to the conditions at collapse resulting from the following arrangements of load on both the pinned- and fixed-base frames :

(1) a vertical load applied at the centre of the beam ;
(2) a horizontal load applied at the top of the stanchion ; and
(3) the most severe combination of the loadings (1) and (2).

For the pinned-base frame the last condition is obtained when the two loads are equal ($V = H$ in *Fig. 3* (*a*)), and for the fixed-base frame when the horizontal load is twice the vertical ($H = 2V$ in *Fig. 4* (*a*)), since under these combinations the whole of the windward half of the beam becomes fully plastic. The bending moment distribution throughout the portal, due to each of the three loadings when sufficient to cause collapse, is represented by the shaded area * in the lower diagram of (*b*)–(*d*) of *Figs 3* for the pinned base frame, and of *Figs 4* for the fixed-base frame. It will be noted that the horizontal base-line of these diagrams represents the total length of the portal when opened out. On the upper diagrams the plastic hinges at the points of full plastic moment are indicated by the closed circles. Assuming a value of 18 tons per square inch as an upper limit for

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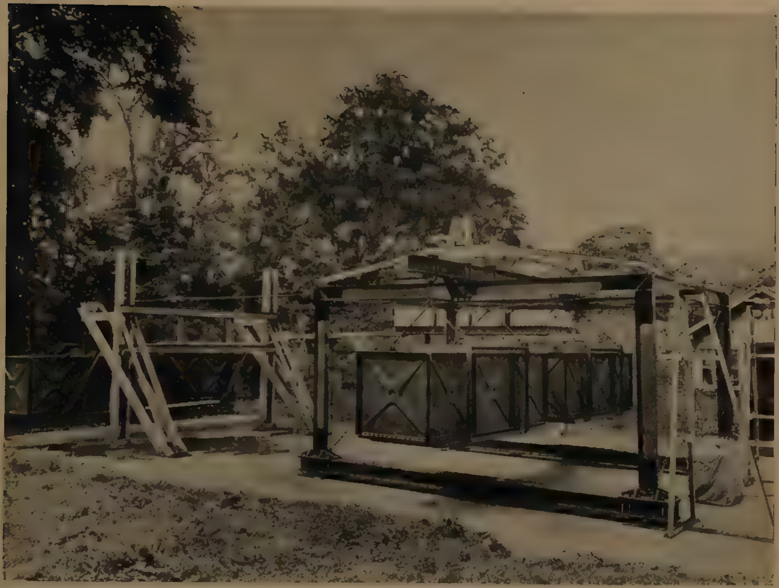
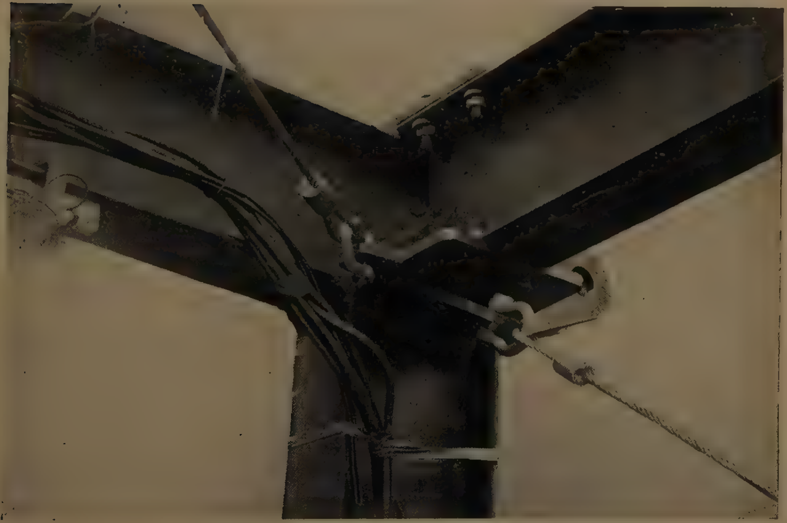
* For details of the construction of these bending moment diagrams, see reference 3, p. 94.

Fig. 2



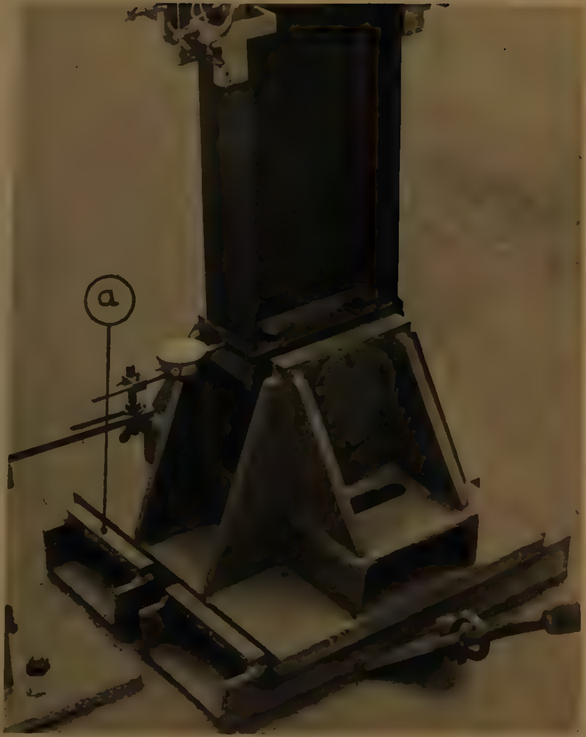
FIXED-BASE PORTALS READY FOR TEST

Fig. 8 (a)



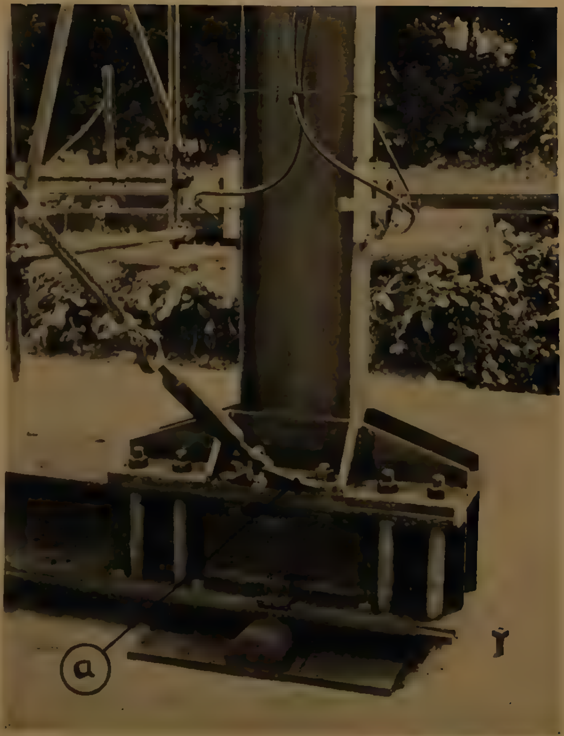
TOP OF STANCHION

Fig. 8 (b)



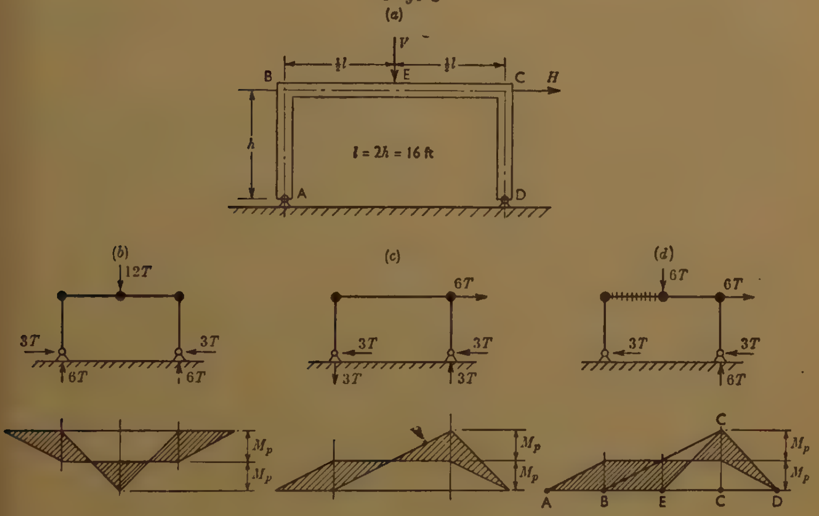
Pinned Base

Fig. 8 (c)



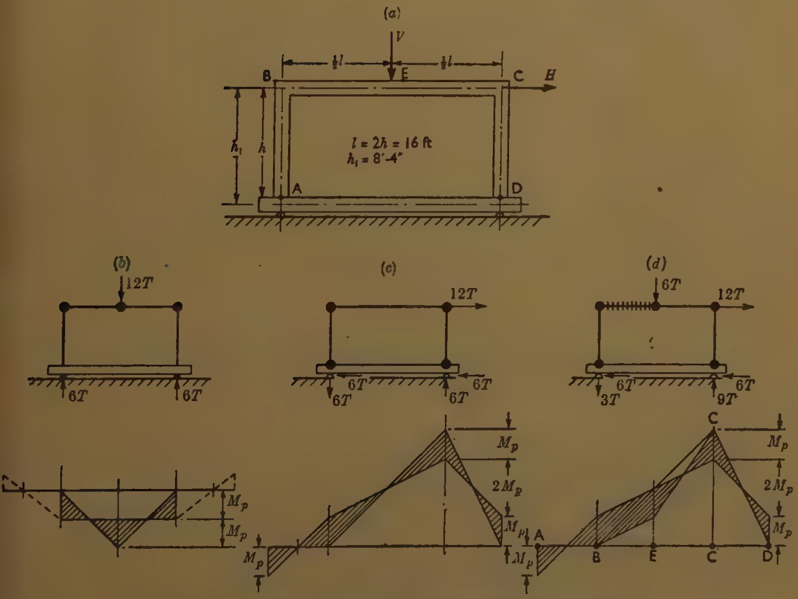
Fixed Base

Figs 3



BENDING-MOMENT DIAGRAMS. PINNED-BASE PORTAL

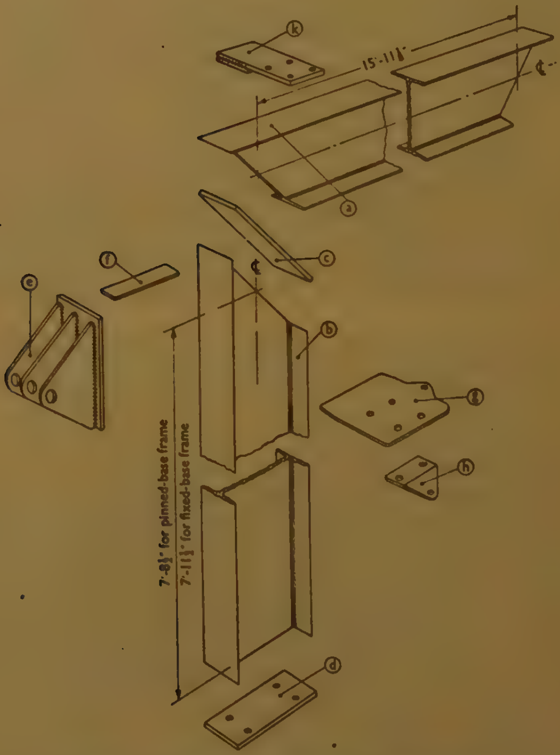
Figs 4



BENDING-MOMENT DIAGRAMS. FIXED-BASE PORTAL the yield stress, the full plastic moment for the 8-inch-by-4-inch joist is 288 tons-inches, the value from which the collapse loads and reactions noted in *Figs 3* and *4* have been determined. All details were designed to withstand these loads.

The horizontal loading frame (G in *Fig. 1*, p. 73) was designed to carry a load of 12 tons in each tank, (*a*) when transmitted through the cables to the portal frame and (*b*) when suspended from the axle of the pulley.

Fig. 5



Details of Beam-to-Stanchion Connexion

Pinned Base Frame

The 8-inch-by-4-inch joist material was cut and machined to the dimensions given for the beam and stanchion members, a and b respectively in *Fig. 5*. The division plates c were cut from 4-inch-by-½-inch bar and the lug e through which the horizontal load was to be applied, was of ½-inch plate and was provided with clearing holes for a 1½-inch-diameter pin. Since this lug was to be welded to the flange of the stanchion, the web at the section in line with the lower flange of the beam, had to be strengthened by the inclusion of ½-inch-thick stiffeners f and g, the latter being extended to form both a seating for the tie beams P and Q (*Fig. 1*), and a small lug for attachment of the wire bracing in the top panel of the frame.

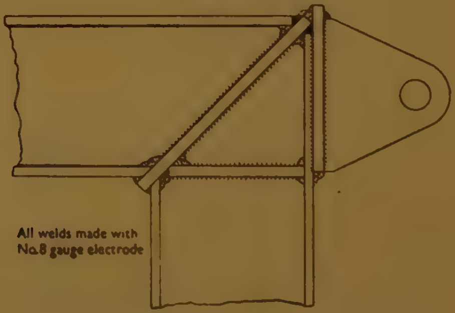
All welding on the stanchions was completed in the shop, namely, the attachment of the division plates c, the base plates d, the stiffeners f and g, and the lug e in the case of the leeward stanchions. The components were then laid out on the concrete site, temporarily braced between stanchion bases, and the web of the beam welded to the division plates. Thereafter, the portal could be turned to complete the other welds in the most convenient positions. Except where otherwise indicated (*Fig. 6*), all welds throughout the frame consisted of a single run of a No. 8 gauge electrode. When all welding had been completed and the temporary bracing removed, the maximum dimensional error was an increase of ½ inch in the distance between the stanchion bases.

It was considered desirable in this first full-scale test to provide stanchion bases approaching as closely as possible to the ideal condition assumed in design. Carefully made pin-base units, shown in *Fig. 7*, p. 79, were therefore provided. These consisted essentially of a main frame a, forming a housing for the pair of M.S.13 ball-races b, the fitting d for bolting up the base of the stanchion being a sliding fit on the 1½-inch-diameter pin c spanning between the two bearings. The whole unit was secured to the concrete foundation by means of the nuts e on two 1⅛-inch-diameter holding-down bolts fitted with base plates and embedded in the concrete to a depth of 18 inches. The channel-shaped packings f were introduced to correct for an alteration made, in the course of construction, in the height of the horizontal loading cables.

The erection of the frames was begun by spreading a thin layer of grout over the concrete around the holding-down bolts, the pin-based units with their channel packings being locked in position. When the grout had set, the nuts were slackened off until the units, by virtue of the slotted holes in the main frame, were quite free on the channels, which themselves were a tight fit on the holding-down bolts. The portals were then raised into position and bolted to the fittings d (*Fig. 7*); the beams P and Q (*Fig. 1*) were landed on the brackets g (*Fig. 5*) and secured by the plates k (*Fig. 5*) which were welded to the portals and bolted to the beams. When the frames had been plumbed and levelled the pin-based units were again locked down, and to make doubly sure, the stops a (*Fig. 8* (*b*), facing p. 75) were welded on. To complete the erection the bracing cables (breaking load 5 tons) were attached to the various lugs as shown in *Figs 8* (*a*) and (*b*) and were brought to a uniform tension by means of strainers.

To support the loading tanks at the centre of the frame, an 8-inch-by-6-inch joist, 20 feet long, was clamped to each of the portal beams by four bolts passing through a strap at the top and a saddle piece at the bottom, as shown in *Fig. 2* (facing p. 74) and *Fig. 9* (*a*). Any restraint of the compression flange of the portal beam was reduced to the minimum by welding to the bottom flange of the 8-inch-by-6-inch joist a length of half-round section at the position of contact. Each of the vertical loading tanks was carried in a cradle suspended from two points on the joist. Each of the other pair of tanks, for the horizontal loading, was supported by a pair of cables having an individual breaking load of 9 tons. These cables were connected to a lug (e in *Fig. 5*) at the top of the stanchion and passed over a double pulley mounted on two heavy ball-races in the frame G (*Fig. 1*, p. 73) and attached to the cradle at a single point. The standard cables available were too short to span the full distance and, since four cables had to be used at each position, the opportunity was taken to insert between each pair, a tension link (H1 and H2 in *Fig. 1*) to which an extensometer could be attached to check the load transmitted.

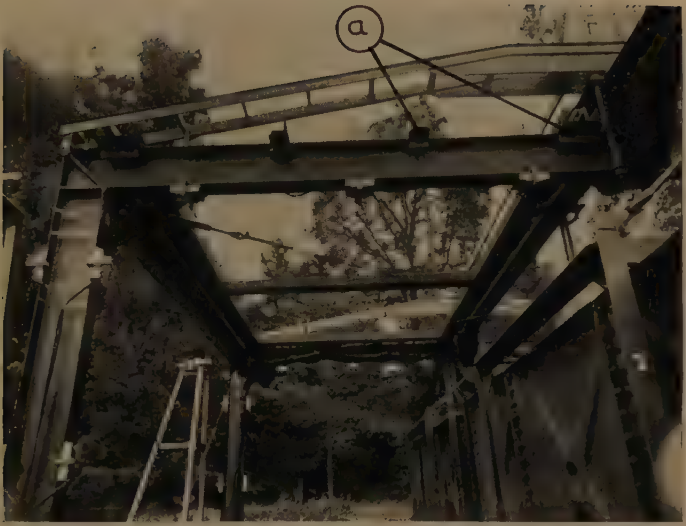
Fig. 6



LEEWARD BEAM-TO-STANCHION CONNEXION

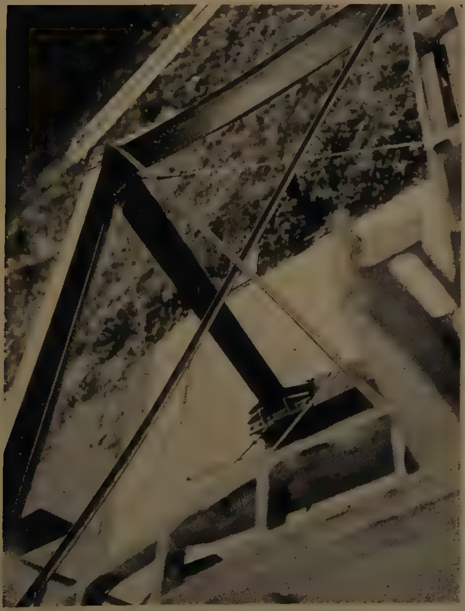
Ultimately each portal was to be subjected to the moment distribution shown in *Figs 3* (*d*), p. 75, according to which the windward half of the beam would have to resist a uniform bending moment of the full plastic value. There was then the possibility that, owing to the considerable plastic yielding, this member might become unstable laterally before developing the full moment of resistance. Some restraint of the lower flange of the beam could be expected from the saddle piece attached to the loading beam (C in *Fig. 1*, p. 73), but this could not prevent lateral movement corresponding to longitudinal displacement of the loading beam. The truss shown at F in *Fig. 1* and in *Fig. 9* (*a*) was therefore mounted on the two portal beams and the close fitting stops (a in *Fig. 9* (*a*)) provided on both sides of the flange. To ensure that the nature of the loading of the frames should be disturbed as little as possible, four brackets were arranged so that the weight of the truss was supported at points approximately on

Fig. 9 (a)



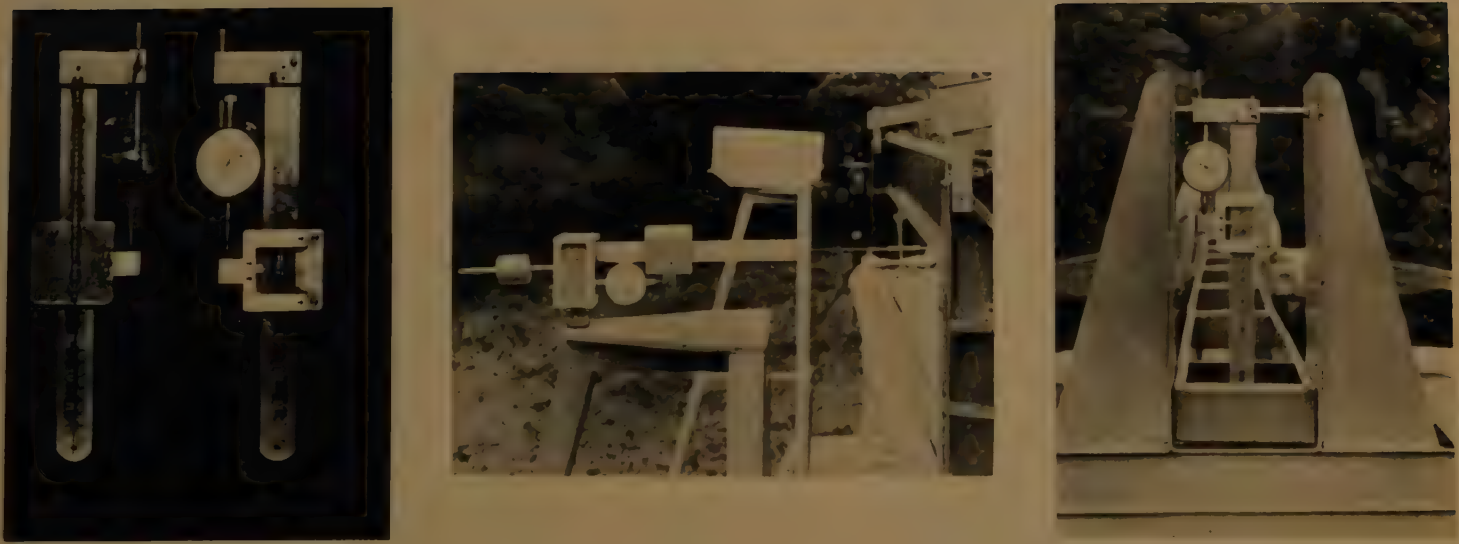
Loading Beams and Stabilizing Truss. Frame FSF.2

Fig. 9 (b)



Diagonal Bracing. Frame FSF.3

Figs 11



(a)

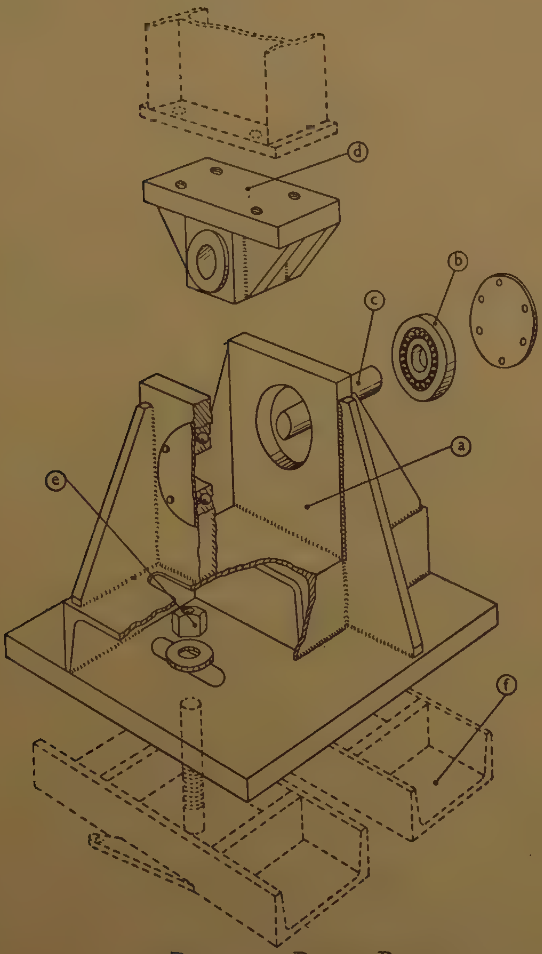
(b)

(c)

DEFLEXION GAUGES

the axes of the windward stanchions and at two other points on the loading beam.

Fig. 7



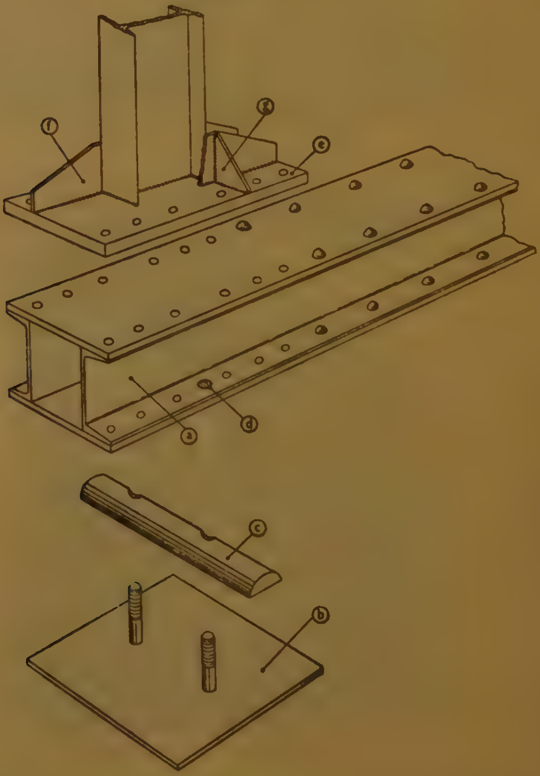
DETAILS OF PINNED BASE

Fixed Base Frames

The construction of the fixed-base frames differed from that of the pinned-base frames only in small details. The lengths of the stanchions had to be increased as indicated in *Fig. 5*, p. 76, the base plates d were omitted and the welding was done without the bracing between the stanchion bases. The resulting distortion was, however, no greater than for the pinned-base portals.

The compound girder (a in *Fig. 10*), to which the stanchion bases were fixed, was made up of two 7-inch-by-3-inch channels with two 12-inch-by-½-inch plates, 18 feet long. This girder was provided in the hope that the restraining moments at the feet of the stanchions could be deduced from its deflexions. Whilst it was capable of developing within its elastic range a moment equal to the full plastic moment of resistance of the stanchion section, it was estimated that, assuming a rigid connexion between stanchion and compound, the horizontal deflexion of the portal would be increased by about 30 per cent as compared with one with fixed feet. Since this girder was purchased as a standard length, it was necessary at each end to remove certain rivet heads, weld in the shanks and drill additional holes to make the connexion shown in *Fig. 8* (*c*), facing p. 75. The

Fig. 10



Details of Fixed Base

plates b (*Fig. 10*) fitted over the holding-down bolts were bedded in grout; next came the length of 4-inch-diameter half-round section and finally the compound girder with the holding-down bolts projecting through the holes d in the botton flange and the base plates e bolted to the top flange. After levelling up, the compounds were locked down and the portal frames erected as before, but with the stanchions resting freely on the base plates e. A single run of weld with a No. 8 gauge electrode was then made around the profile of the joist section at each base, two extra runs being added on the outside of each flange. The plates f and g were provided solely to stiffen the base plate, and were purposely not connected to the stanchion, which was held only by the welds around the end profile. Finally, after attaching the lugs a (*Fig. 8* (*c*), facing p. 75), the erection was completed by fitting the various straining cables and bringing them all to a uniform tension.

The erection of the loading gear, including the truss F (*Fig 1*, p. 73), was carried out in the same way as for the pinned-base frame.

Instruments

The essential requirement of the gauges for measuring deflexion was that they should record the small elastic and somewhat larger plastic deflexions with the same degree of accuracy. This was achieved by combining in one instrument the two types of gauges shown in *Fig. 11* (*a*), facing p. 79, namely, a standard dial gauge of 1-inch travel graduated in thousandths of an inch and a 12-inch rule with vernier reading to 0·01 inch. The latter piece, while free to slide, was held firmly against the rule by a spring plunger and attached to the end block by a length of linen-covered elastic to accommodate large deflexions without undue resistance. The dial gauge was clamped in the end block with its plunger in contact with the vernier to which was connected a length of fine wire for attachment to the point at which the deflexion was required.

The detail for the mounting of these gauges in the frames E and on the stands D (*Fig. 1*, p. 73) for recording vertical and horizontal deflexion respectively, can be seen in *Figs 11* (*b*) and (*c*), facing p. 79.

For the recording of strains, complete Maihak acoustic strain-gauge equipment was kindly lent by the Department of Aeronautical and Engineering Research of the Admiralty, and comprised one hundred and fifty 10-centimetre gauges, a bank of push-button selector switches to accommodate two hundred gauges, together with the necessary leads and a receiver set.

As a preliminary to the frame tests, all gauges were calibrated on a simple tension bar, the modulus of elasticity of which had previously been determined using a pair of mirror-type extensometers.

The gauges were attached at twelve stations on each portal frame; in the case of the stanchions, at 4 feet from the centre-line of the beam, and at 2 feet 6 inches on either side of this position; and for the beams, at the quarter points and at 2 feet 6 inches on either side. The arrangement of the gauges at a station near the base of a stanchion can be seen in *Fig. 8* (*c*), facing p. 75, four gauges being used so that the bending moments about both the major and minor axes could be obtained.[6] Each gauge was held in position by a single clamp. The terminals of the exciting coils were connected to the selector switches at L (*Fig. 1*, p. 73) and these in turn, to the receiver set M in the hut at N.

The only difficulty experienced in using the gauges was that with the longer lengths of lead the gauge wire could not be made to vibrate satisfactorily. But, after trial and error it was found that a resistance of 2,200 ohms between earth and one set of gauge leads to cut out high-frequency oscillations was sufficient to make all gauges operate satisfactorily, the maximum length of lead being of the order of 20 yards.

DESCRIPTION OF TESTS

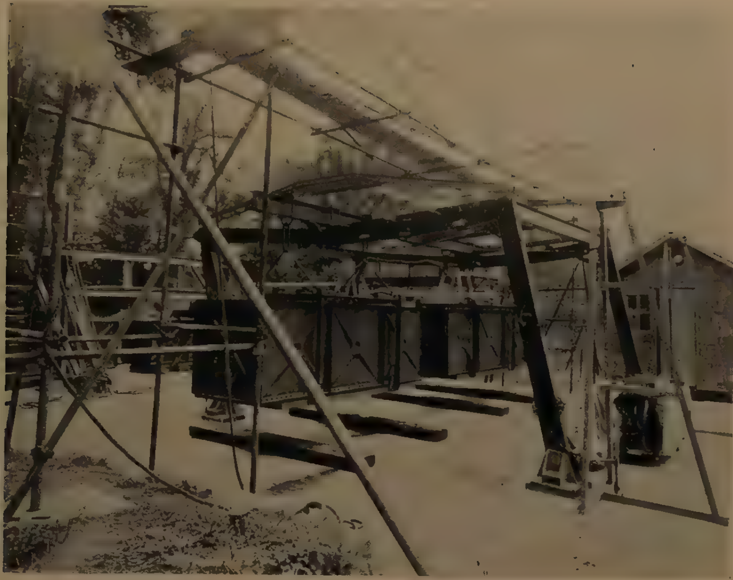
Pinned Base Frame F.S.F.1

Before the final loading to collapse, preliminary data were obtained from a few tests within the elastic range using water loading only. The observed deflexions and the moments deduced from the observed strains for vertical and horizontal loads when acting separately, or as a combined loading, were in good agreement with those calculated on the simple elastic theory.

As already mentioned, it was proposed in the test to collapse to apply equal horizontal and vertical loads since, according to the simple theory, this arrangement produces at collapse the maximum degree of plasticity, the whole of the windward half of the beam being subjected to the full plastic moment as indicated in *Figs 3* (*d*) p. 75. The greater part of the loading had now to be made up with track links and it was felt that some precaution should be taken to ensure that the task of loading the tanks did not become dangerous owing to some unforeseen failure. The scaffolding shown in *Fig. 12* (*a*), erected well clear of the test frame, was therefore provided for lowering track links into the vertical loading tanks. This was, of course, considerably slower than loading with water and the test had to be spread over 2 days.

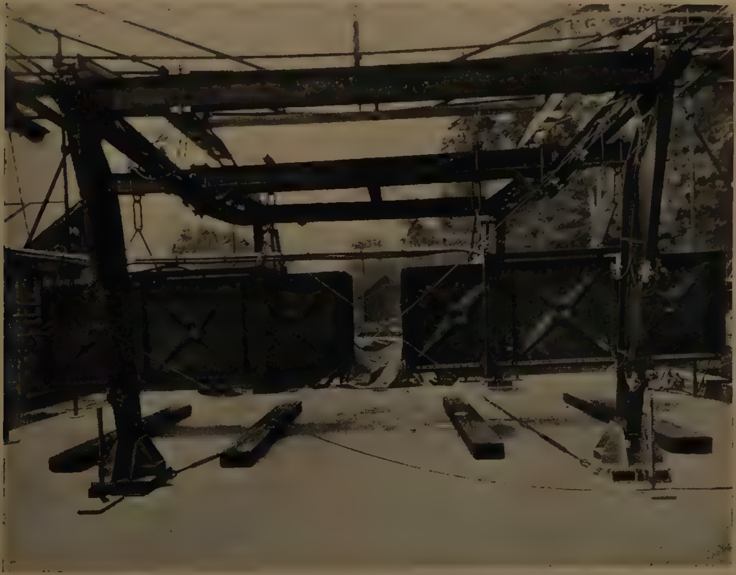
On the first day, the aim was to cover the greater part of the elastic range. The loads were applied in increments of 0·5 ton per tank up to a total loading of $V = H = 3{\cdot}13$ tons, estimated to produce a maximum stress of 14·86 tons per square inch. At the end of this range a number of the strain gauges were nearing the limit of their extension, and the first task the following day was therefore to reset all gauges and check the operation of the equipment. A complete set of readings were then taken and the loading continued in increments of 0·25 ton per tank. When several increments had been added, definite signs of creep were observed and thereafter readings were not taken until the rate of creep had fallen to about 0·01 inch per minute on the horizontal gauge. At 4·88 tons the

Fig. 12 (a)



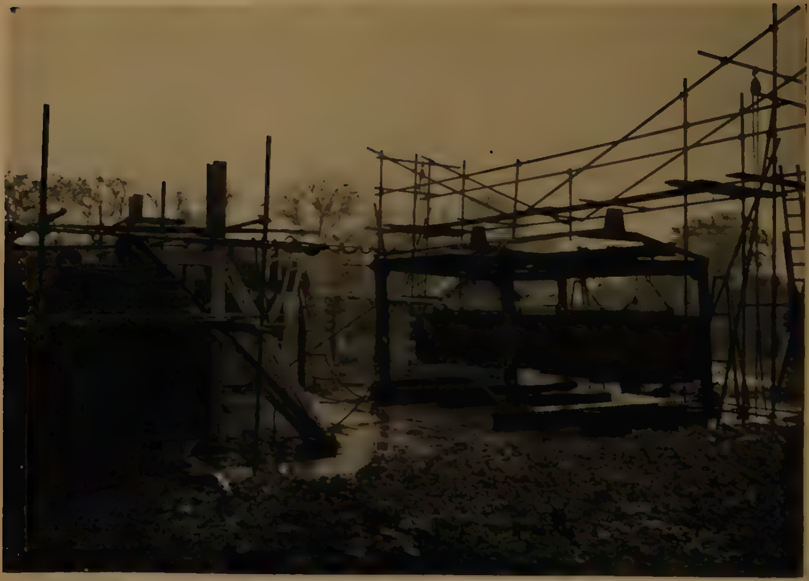
Side View of Frame FSF.1 at Collapse

Fig. 12 (b)



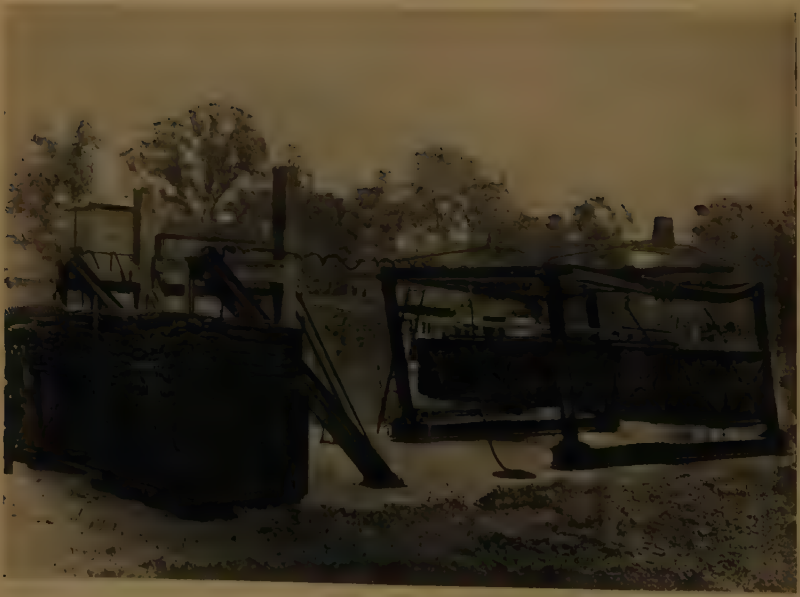
End View of Frame FSF.1 at Collapse

Fig. 17



Fixed-Base Frame FSF.2 under Final Load

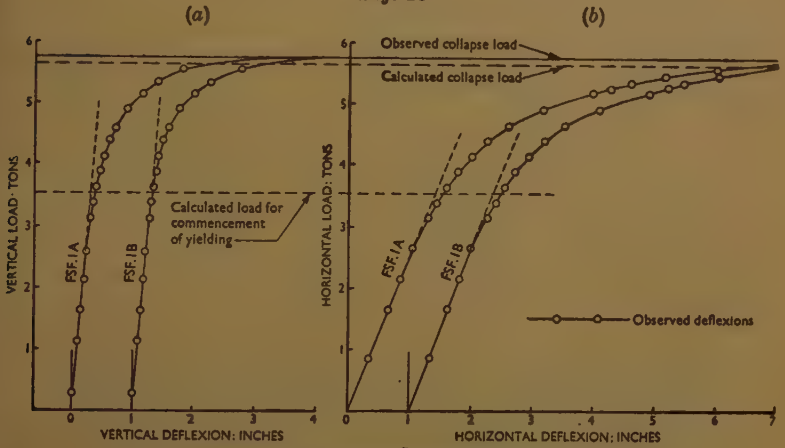
Fig. 20



Frame FSF.3 after Collapse

set of strain gauges on either side of the joint C (*Fig. 3* (*a*)) again reached their limit and had to be removed. These were followed shortly afterwards by the gauges at the top of the other stanchions and those in the length BE. At 5·15 tons, water loading was started and the increment reduced to 0·10 ton, though readings were taken only after alternate increments. At 5·75 tons the sideway deflexion was observed to be increasing rapidly and after about 3 minutes the frame collapsed, the horizontal tanks descending on to the timber sleepers provided to break the impact on the concrete foundation. The form of the frame after collapse can be seen in *Figs 12.* As the frame was failing it was observed to move rapidly through a distance of approximately 1 foot in the horizontal direction before the leeward stanchions became unstable and deflected laterally as shown in *Fig. 12* (*b*).

Figs 13



Observed Deflexions—Frame FSF.1

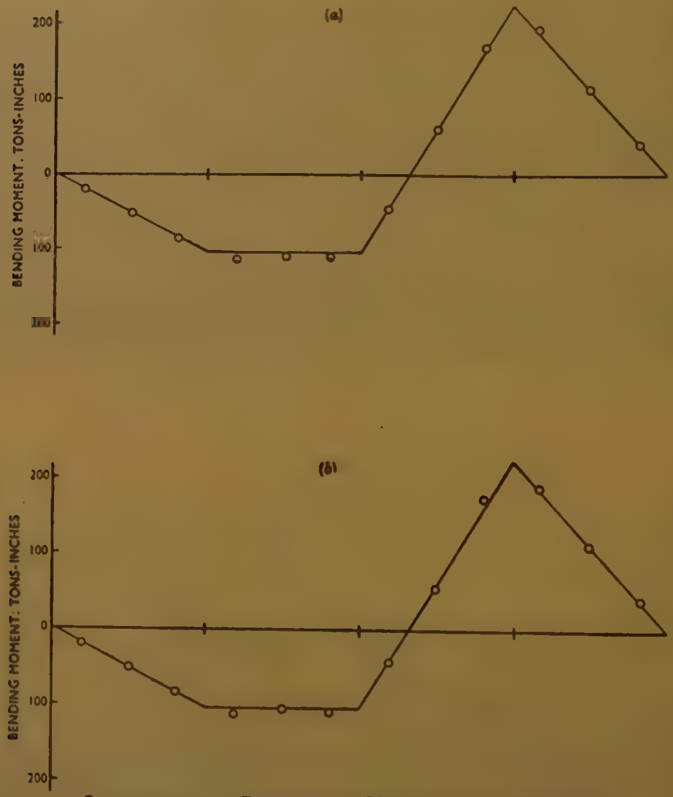
The observed deflexions have been plotted in *Figs 13* and compared with calculated values in the elastic range. The latter are shown in broken line. The symbols FSF.1A and FSF.1B in *Figs 13* refer to the individual portals, the former denoting the one nearest to the hut N (*Fig. 1*, p. 73). Observed bending moments have also been compared with theoretical values; these are plotted in *Figs 14* for the loading 3·40 tons, the first load applied on the second day.

Fixed-Base Frames FSF.2 and 3

Frame FSF.2.—In view of the good agreement between observed and calculated data in the previous tests it was not thought necessary to examine the behaviour of this frame under the individual loads.

A preliminary test was carried out using the same arrangement of loading as it was proposed to apply in the final test, namely, a vertical load and a horizontal load of twice the value. Owing to the larger horizontal load, it was not possible to apply a sufficient range of loading using water only. Hence for the first loading, the horizontal loading tanks were each made up to 2·25 tons with scrap metal to equal twice the load applied by an empty vertical loading tank. A second and a third loading were obtained by running in water in increments of 0·5 ton to the vertical tanks and of 1·0 ton to the horizontal tanks.

Figs 14

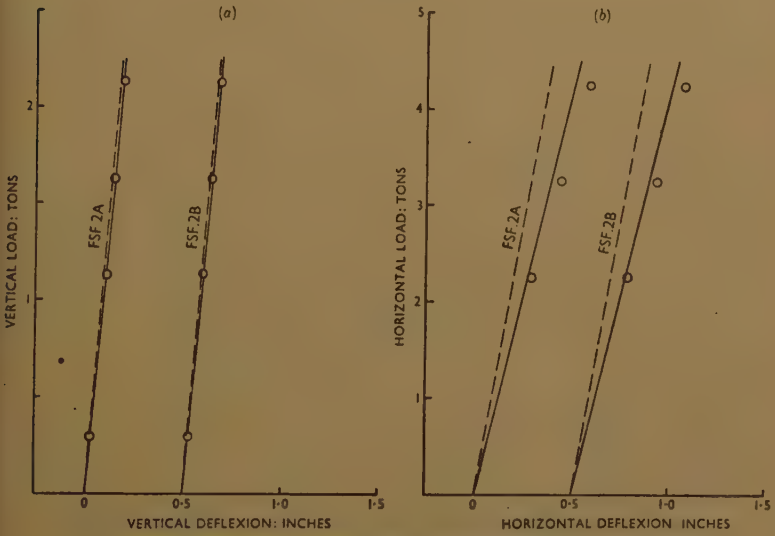


Observed and Calculated Moments. Frame FSF.1
($V = H = 3{\cdot}40$ tons)

The observed deflexions and the bending moment due to the maximum loads are plotted in *Figs 15* and *16* respectively. Two sets of calculated values are given in each case. For the first calculation it has been assumed that the stanchion bases were rigidly held, while in the second case, account has been taken of the stiffness of the compound girder assuming it to be rigidly connected to the portals, but simply supported on the concrete as shown in *Fig. 4* (*a*).

The agreement of observed moments and deflexions with the second set of calculated values was considered satisfactory, though the gradual deviation of the observed horizontal deflexions, and the somewhat lower values of observed bending moments near the bases of the stanchions, indicated a relaxation of the connexion between them and the compound girders. This was later confirmed by measurements of rotations at the stanchion bases.

Figs 15



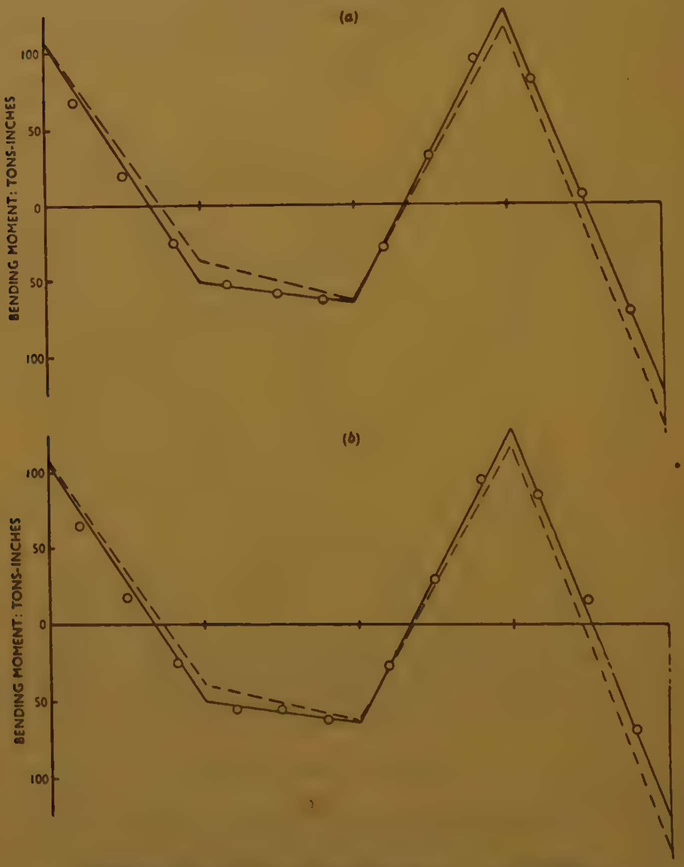
OBSERVED AND CALCULATED DEFLEXIONS. FRAME FSF.2

(Broken lines show deflexions calculated assuming rigid bases; full lines show deflexions calculated allowing for stiffness of compound girder)

The method of carrying out the final test was similar to that for the pinned-base frame. On the first day the test was taken to the stage $V = 3{\cdot}63$ tons, $H = 7{\cdot}26$ tons, the loading being applied in increments of 0·5 ton to the vertical tanks and of 1·0 ton to the horizontal tanks. The calculated maximum stress under these loads was 15·75 tons per square inch. On the second day the increments were reduced by half and loading continued until $V = 5{\cdot}13$ tons, $H = 10{\cdot}26$ tons when water loading was started by increments of 0·1 and 0·2 ton in the vertical and horizontal tanks respectively.

Since this frame had been made up from the same batch of material as the pinned-base one, it should, according to the simple plastic theory, have collapsed under the same vertical load, namely $V = 5{\cdot}75$ tons. However, apart from a slight increase in the increments of deflexion, no signs of catastrophic failure could be detected. After four more increments of load had been added for almost constant increments of deflexion, bringing the loading to $V = 6{\cdot}13$ tons, $H = 12{\cdot}26$ tons, the test was discontinued for the day. The next morning all gauges were re-read and the horizontal deflexion was found to have increased over night by about 18 per cent. A further reading some hours later showed no further increase. Additional loading was then applied but had to be discontinued

Figs 16



Observed and Calculated Moments. Frame FSF.2
($H = 2V = 4{\cdot}25$ tons)
(Broken lines show moments calculated assuming rigid bases; full lines show moments calculated allowing for stiffness of compound girder)

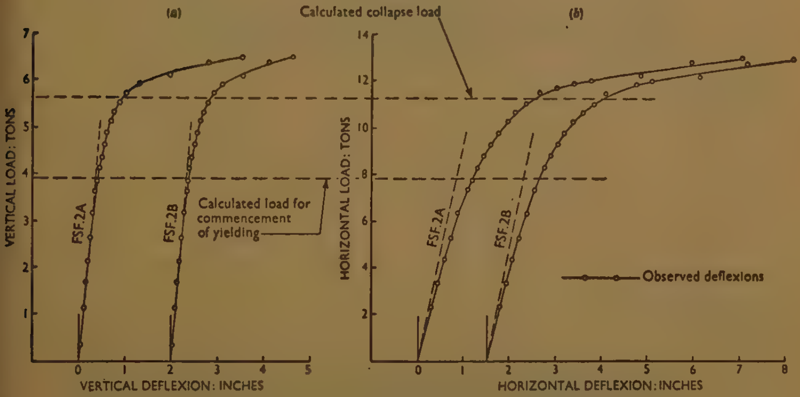
at $H = 13{\cdot}11$ tons, the limiting capacity of the horizontal tanks. The load was left on for 3 days but showed no signs of causing collapse.

The state of the frame under the final load can be seen in *Fig. 17* (facing p. 83). The observed deflexion and bending moments have been plotted in *Figs 18* and *19* respectively. The calculated values in each case were obtained by taking account of the stiffness of the compound girder as described for the preliminary test, though the load at collapse is, of course, independent of this stiffness.

Records of the strains in the tension links in the horizontal loading cables showed that the frame had been subjected to the full loads.

Frame FSF.3.—Though this test was intended to be a repetition of test FSF.2, the opportunity was taken to examine the effect of certain modifications, some of which tended to lessen the precautions previously taken against premature failure. The welding at the stanchion bases was reduced by omitting the extra runs on the outsides of the flanges. Also, in the course of erection, the truss F (*Fig. 1*, p. 73), used in the first two tests to stabilize the compression flanges of the windward halves of the portal beams, was replaced by the simple diagonal bracing illustrated in *Fig. 9* (*b*), facing p. 78, which shows the frame after collapse. This, while sufficient to prevent longitudinal displacement of the loading beam (C in *Fig. 1*), left the beam of each portal unsupported for a length approximately 21 times the width of the flange.

Figs 18



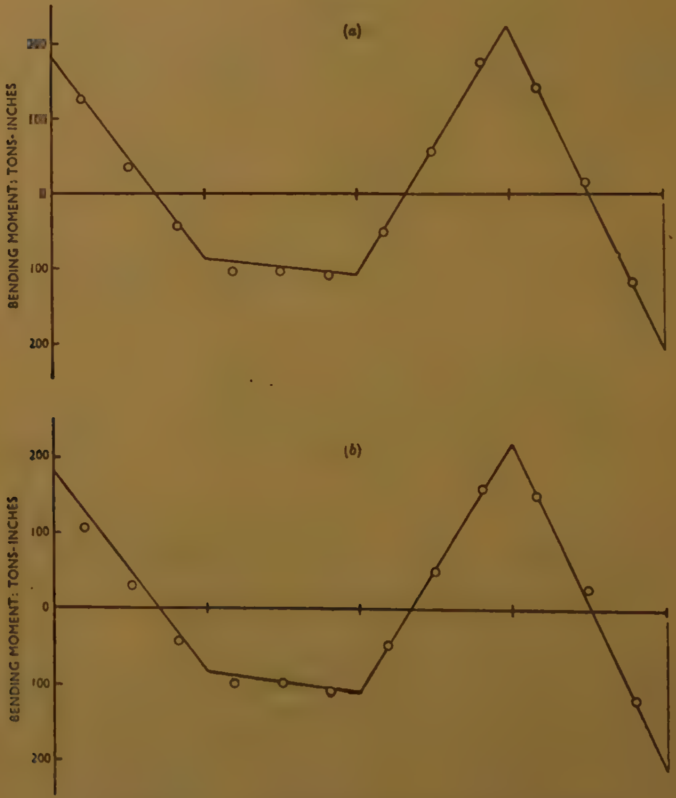
Observed Deflexions. Frame FSF.2

In view of the good agreement in the observed data obtained from the pair of portals in previous tests, it was decided to fit Maihak gauges to portal FSF.3A only.

As a preliminary, the frame was subjected to several applications of the loading $V = 1{\cdot}57$ tons, $H = 3{\cdot}13$ tons, and the total deflexions were recorded. It was found that these could be repeated consistently and were in good agreement with those observed for frame FSF.2. The behaviour of the frame was therefore considered satisfactory.

For the final test, the beam-to-stanchion connexions, portions of the adjacent members, and the stanchion bases on portal FSF.3B were coated with plumbers' resin, as shown in *Fig. 2*, (facing p. 74), so that the development of yielding could be easily followed. On the first day the loading was taken to the stage $V = 3{\cdot}07$ tons, $H = 6{\cdot}13$ tons, in increments of 0·5 ton and 1·0 ton in vertical and horizontal tanks respectively. On the second day the increments were reduced by half. The first signs of cracking in the resin were observed on the inside flange at the top of the leeward stanchion

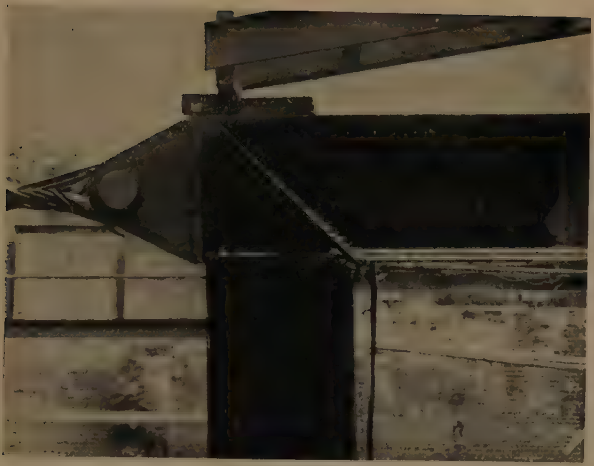
Figs 19



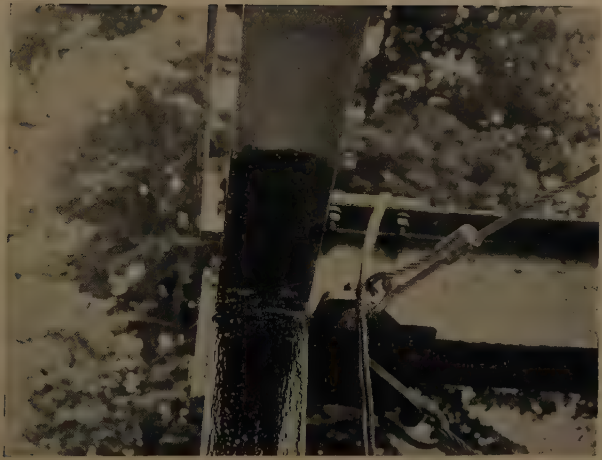
Observed and Calculated Moments. Frame FSF.2
($H = 2V = 7{\cdot}26$ tons)

Figs 21

(*a*)



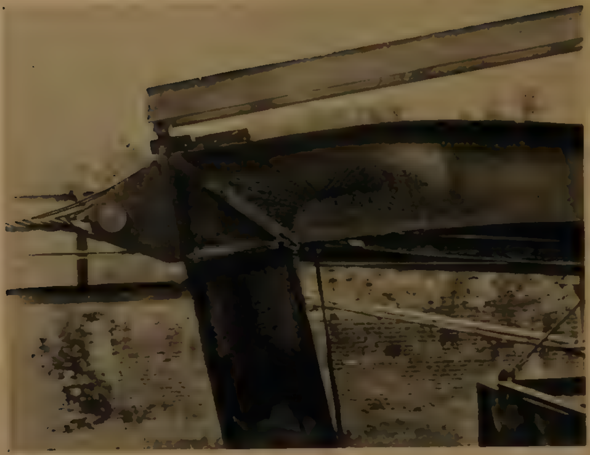
(*b*)



Development of Yielding. Portal FSF.3B
($V = 4{\cdot}82$ tons; $H = 9{\cdot}63$ tons)

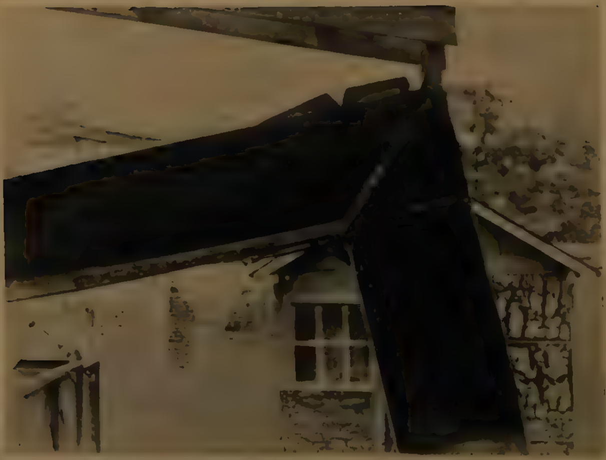
Figs 21

(c)



LEEWARD CONNEXION

(d)



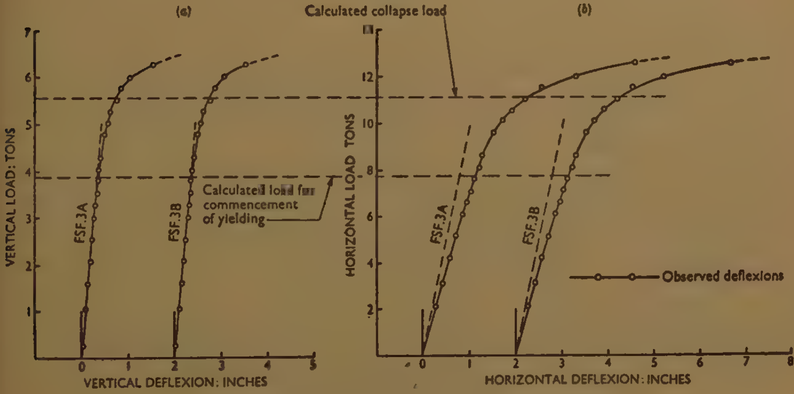
WINDWARD CONNEXION

PORTAL FSF.3B, AFTER COLLAPSE

at a load of $V = 3{\cdot}82$ tons. At $V = 4{\cdot}82$ tons this yielding had only advanced to the stage shown in *Fig. 21* (*b*), the shear cracks which had begun to appear in the web of the connexion being shown at (*a*). At the same load, cracks were also observed in the resin on the tension flanges near the bases of both stanchions. Thereafter, yielding could be seen to be developing at each of these three positions, though at the windward beam-to-stanchion connexion the resin remained almost undisturbed until the end of the test.

After adding the first increment of water to bring the load up to $V = 6{\cdot}32$ tons and taking a set of readings, there was an ominous cracking sound caused by the breaking of the weld at one of the stanchion bases, followed immediately by the fracture of the other three to produce complete collapse.

Figs 22



OBSERVED DEFLEXIONS. FRAME FSF.3

The condition of the frame after collapse can be seen in *Fig. 20* (facing p. 83).

The observed deflexions for the two portals are shown in *Figs 22* and the observed bending moments for the portal FSF.3A at the loading $V = 3{\cdot}57$ tons in *Fig. 23*. The calculated values were obtained in the same way as for the previous frame.

COMPARISON OF OBSERVED AND THEORETICAL CARRYING CAPACITIES

As part of this investigation a number of joists taken from the batches of 8-inch-by-4-inch material used in the construction of the frames, had previously been tested as simply supported beams.

Four joists out of the first batch from which frames FSF.1 and 2 were prepared, gave a minimum average yield stress for the section of 16·9 tons per square inch. Thus, according to the simple plastic theory, yielding in the extreme fibres starts when the bending moment reaches a value of:

$$M_y = fZ = 236 \text{ tons-inches}$$

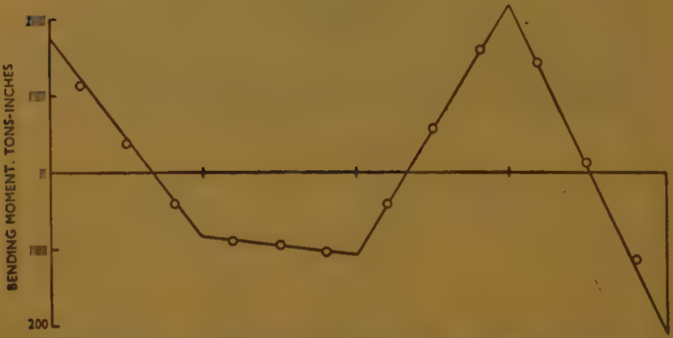
and penetrates through the full depth when the moment has the value:

$$M_p = 1{\cdot}15M_y = 271 \text{ tons-inches}$$

Referring to *Fig. 3* for the pinned-base frame FSF.1, the maximum moment ($66H$) in the elastic range occurs at C so that the calculated load for commencement of yielding is:

$$H = V = 3{\cdot}58 \text{ tons.}$$

Fig. 23



OBSERVED AND CALCULATED MOMENTS. FRAME FSF.3A
($H = 2V \doteq 7{\cdot}41$ tons)

At collapse the theoretical distribution of moment is that shown in diagram (*d*), the ordinate Cc representing a moment of Hh where

$$Hh = 2M_p,$$

so that, for the above value of M_p, collapse should occur at a load of

$$H = V = 5{\cdot}65 \text{ tons.}$$

These values are shown on the load deflexion curves in *Figs 13*, p. 83, and the latter will be seen to be in good agreement with the observed collapse load.

For the fixed-base frame FSF.2, the maximum elastic moment ($30{\cdot}17H$) again occurs at C (*Figs 4*, p. 75) and yielding commences at a load of

$$H = 2V = 7{\cdot}82 \text{ tons.}$$

At collapse, since the ordinate Cc (diagram (*d*), *Figs 4*) has the value Hh, and

$$Hh = 4M_p$$

the corresponding loading is

$$H = 2V = 11{\cdot}3 \text{ tons.}$$

These values are shown in *Figs 18*, p. 87. Though this theoretical collapse load was exceeded by a considerable margin in the test, it gives a good indication of the stage beyond which the frame developed large permanent deformations.

Frame FSF.3 was made up from a second batch of material for which tests on four joists gave the minimum average yield stress of 16·8 tons per square inch so that

$$M_y = 234 \text{ tons-inches}$$

$$\text{and } M_p = 269 \text{ tons-inches.}$$

Thus, as for FSF.2, the calculated yield loading is:

$$H = 2V = 7{\cdot}75 \text{ tons,}$$

and the collapse loading:

$$H = 2V = 11{\cdot}20 \text{ tons,}$$

both of which are indicated in *Figs 22*, p. 89. These results will be seen to be in good agreement with those for the previous frame.

In the tests on the simply supported joists it was found that under symmetrical two-point loadings where a portion of the member was subjected to pure bending, collapse occurred at a moment in close agreement with the theoretical value M_p. When, however, a single concentrated load was applied, the extent of the plastic yielding was considerably reduced and within the zone high strains were very soon developed causing strain hardening before large vertical deflexions had been recorded. Thereafter, the moment of resistance of sections in the plastic zone began to build up and to exceed the value M_p so that it was not possible to produce collapse of the joint by bending only.

In consequence of the loading arrangements selected for the frames, both conditions of yielding were known to have occurred in both the pinned- and fixed-base frames. In the former, the length BE (*Fig. 3* (*a*), p. 75) must eventually have been subjected to a uniform moment of the full plastic value, while at the top of the stanchion CD yielding developed in the same way as for the simply supported joist carrying a single concentrated load. Since the moment at the latter position could exceed the value M_p owing to strain-hardening, the frame should have carried a load in excess of the calculated value of $V = H = 5{\cdot}65$ tons. The fact that it collapsed at a load in close agreement with this value can, however, be explained from a consideration of the deformation of the frame. From statical considerations it will be seen that the whole of the vertical load V is reacted at D so that if, for a given increment of load, the frame sways over by an amount δ, there will be an additional increase of moment at C of $V\delta$. Obviously, when the rate of growth of these sway moments approaches that of the added resistance caused by strain-hardening, collapse must occur.

It will be noted from *Fig. 13* (*b*), p. 83, that at the last recorded deflexion

($V = H = 5{\cdot}55$ tons) the total sway moment had reached a value of 33·1 tons-inches, which is more than ten per cent of the full plastic moment of resistance of the joist.

For the fixed-base frames, the horizontal deflexions at corresponding stages were considerably less than those observed for the pinned-base frame (compare *Figs 13* and *18*). The consequent reduction in the sway moment, combined with the fact that in this case strain hardening also occurred at the base of the stanchion, was evidently sufficient to prevent catastrophic failure.

SUMMARY AND CONCLUSIONS

(1) The elastic behaviour of all three frames as judged by the observed deflexions (*Figs 13*, *18*, and *22*) and the moments deduced from the strain-gauge readings (*Figs 14*, *19*, and *23*) agreed closely with that forecast by elastic analysis. This was particularly true of the pinned-base frame FSF.1. The provision of compound girders to fix the stanchion bases of frames FSF.2 and 3 was estimated to cause an increase of about 30 per cent in the horizontal deflexion as compared with that which would have occurred had these bases been rigidly held (*Fig. 15*). It was found, however, that as the loading increased, this estimated deflexion was gradually exceeded, though the deviation was comparatively small. Measurement of rotations at the bases of the stanchions also indicated some relaxation in the connexion between stanchion and compound.

(2) Theoretical collapse loads, calculated according to the simple plastic theory were obtained for all three frames, the values of the full plastic moment being based on average yield stresses previously determined for the joist material. For the pinned-base frame the agreement between observed and calculated collapse loads was exceedingly close. In the case of the fixed-base frames, however, the calculated values were exceeded, and for frame FSF.2 it was not possible to produce collapse even when the load had been increased by 18 per cent. The calculated load for frame FSF.3 was also exceeded, though collapse eventually occurred owing to the fracture of light welds at the stanchion bases.

Earlier tests in which lengths of the joist material had been tested as simply supported beams, indicated that real collapse could only occur when a finite length of the member was subjected to a uniform bending moment equal to the full plastic value. Under other conditions of bending, such as those arising from a single concentrated load, strain hardening set in before large vertical deflexions had been developed and so prevented collapse. Since in all three tests this condition occurred in the stanchions, it could be argued that the calculated collapse load should have been exceeded for all three frames. Such an argument, however, takes no account of the effect of the deformations of the frame on the carrying capacity. The side-sway deflexion in conjunction with any vertical reaction at the stanchion base will produce an additional moment at the top of this member. It is therefore claimed that the pinned-base frame collapsed as a result of instability when the rate of increase of the sway moments approached that of the increase in resistance due to strain hardening.

Although in the case of the fixed-base frames the theoretical collapse loads were exceeded, it will be seen from the load/deflexion curves (*Figs 18* and *22*) that the calculated load gives a good indication of the stage beyond which the frame developed large permanent deformations.

(3) In the design of any frame by the plastic method, it is necessary to ensure that types of failure other than by bending about the major axis, do not produce premature collapse. The principal form of failure to be guarded against is lateral instability. It is known that for beam members subjected to uniform bending moment, adequate protection is provided if the distance between lateral supports does not exceed about 100 times the minimum radius of gyration. In frame FSF.3 the length BE (*Fig. 4*) was unsupported for a length 114 times its minimum radius of gyration and it eventually withstood a moment in excess of the full plastic value without showing any signs of becoming unstable laterally.

Another source of danger is in the design of details such as beam-to-stanchion connexions and the fixed bases of stanchions which must be capable of transmitting the full plastic moment of the adjoining members, preferably without undue deformation. The mitred connexion with a division plate used in these tests, was very satisfactory (*Figs 21* (*c*) and (*d*)). For the fixed stanchion bases it was found that even with a very light weld, consisting of a single run of a No. 8 gauge electrode, around the complete profile of the 8-inch-by-4-inch stanchion section (frame FSF.3), a moment equal to the full plastic value for this member could be developed, though fracture occurred shortly afterwards. With two additional runs of weld on the outside of each flange (frame FSF.2) there was no sign of fracture even when the frame carried a load 16 per cent in excess of the theoretical collapse value.

ACKNOWLEDGEMENTS

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The Authors also thank the Department of Scientific and Industrial Research and the British Welding Association whose continued support has made this work possible.

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The Paper is accompanied by seventeen photographs and fifteen sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared.

Paper No. 5832

"Secondary Stresses in Bridge Trusses"

by

Professor Shu-Tao Chen

(Ordered by the Council to be published with written discussion) †

INTRODUCTION

SECONDARY stresses in steel bridge trusses can be analysed by the slope-deflexion method or by the moment-distribution method.[1, 2] Neither of these methods presents any difficulties except in the assumptions made in the first approximations. When the slope-deflexion method is used, the joint-rotation at the far and near ends of a member are usually assumed to be equal. In the moment distribution method, however, it is assumed that the far end joint-rotation is equal to zero. Neither of these assumptions is satisfactory, and at least three cycles of approximations are usually required for ordinary purposes. It is the object of this Paper to show that an assumed value can be assigned to: (*a*) the joint-rotation at; and (*b*) the carried-over moment from, the far end of a member. Two cycles of approximations give the same accuracy as by the conventional method.

ASSUMED JOINT-ROTATION

It is sufficiently accurate to assume that the rotation (slope) of a joint is equal to the average rotation (deflexion) of two members at the joint which have comparatively larger stiffness factors. If only one member is comparatively stiffer, the rotation of that member can be assumed to be equal to the rotation of the joint. Comparatively larger stiffness factors in more than two members seldom occur. The term "comparatively" has no definite meaning, but, in general, it is used by the Author if the stiffness factor of a member is nearly equal to twice that of the member with which it is being compared. Usually the chords including the end posts are such members.

Notation

K denotes the stiffness factor of a member.

$\theta = 2E \times$ rotation (slope) of the joint, which is considered to be positive when clockwise.

[1, 2] The references are given on pp. 103, 104.

† Correspondence on this Paper should be received at the Institution by the 15th May, 1952, and will be published in Part I of the Proceedings. Contributions should be limited to about 600 words.—SEC. I.C.E.

$R = 2E \times$ rotation (deflexion) of the member, which is considered to be positive when clockwise.

The slope-deflexion equations for a joint n are :

$$K_{nf_1}(2\theta_n + \theta_{f_1} - 3R_{nf_1}) + K_{nf_2}(2\theta_n + \theta_{f_2} - 3R_{nf_2}) + K_{nf_3}(2\theta_n + \theta_{f_3} - 3R_{nf_3}) + K_{nf_4}(2\theta_n + \theta_{f_4} - 3R_{nf_4}) = 0$$

If K_{nf_1} and K_{nf_2} are comparatively stiffer (that is to say, if $K_{nf_1} = K_{nf_2} > 2K_{nf_3} > 2K_{nf}$), then, by assuming

$$K_{nf_3} = K_{nf_4} = 0\,;\ \theta_{f_1} = \theta_{f_2} = \theta_n\,;\ \text{and } K_{nf_1} = K_{nf_2},$$

it can be shown that

$$\theta_n = \tfrac{1}{2}(R_{nf_1} + R_{nf_2}) \quad . \ . \ . \ . \ . \ . \ . \ . \quad \text{(A)}$$

Similarly, if only K_{nf_1} is comparatively stiffer (that is to say, $K_{nf_1} > 2K_{nf_2} > 2K_{nf_3} > 2K_{nf_4}$), then, assuming $K_{nf_2} = K_{nf_3} = K_{nf_4} = 0$ and $\theta_{f_1} = \theta_n$ gives :

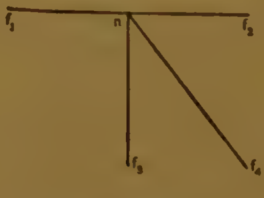
$$\theta_n = R_{nf_1} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad \text{(B)}$$

Since the above two formulae are only approximately true, it is unnecessary to assign $(R_{nf_1} + R_{nf_2})/2$ or R_{nf_1} to be exactly the assumed value of θ_n. The nearest rounded values should be used. Such values can be estimated mentally

Assumed Carried-Over Moment

In contrast to the conventional method, a set of assumed moments is carried over from the far ends to the near ends of the members before the unbalanced fixed-end moments at the joint are distributed. The magnitude of the assumed carried-over moment at the near end of a member is equal to the product of the stiffness factor of the member, the assumed joint-rotation at the far end, and E. Thus in *Fig. 1*, the assumed carried-over moment from n to f_1 is :

Fig. 1



$$K_{nf_1}\theta_n = K_{nf_1}(R_{nf_1} + R_{nf_2})/2 \quad . \ . \ . \ . \ . \ . \ . \quad \text{(C)}$$

if K_{nf_1} and K_{nf_2} are comparatively stiffer ; or the assumed carried over moment from n to f_1 is :

$$K_{nf_1}\, R_{nf_1} \quad . \ . \ . \ . \ . \ . \ . \ . \quad \text{(D)}$$

if K_{nf_1} only is comparatively stiffer. It is again unnecessary to assign $K_{nf_1}(R_{nf_1} + R_{nf_2})/2$ or $K_{nf_1}\,R_{nf_1}$ to be exactly the assumed carried-over moment, and the nearest rounded values should be used. Such values can be estimated mentally.

Order of Solution

Alternate joints are divided into two groups. The sub-joints, in trusses with subdivided panels or with double-web systems, form a third group.

Fig. 2

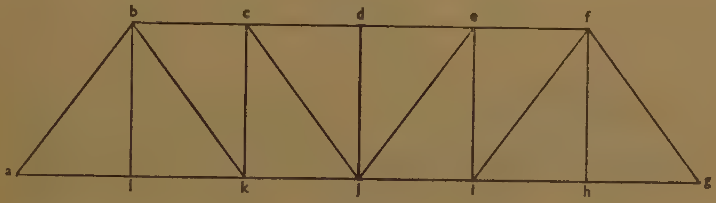
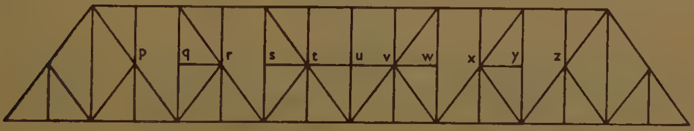


Fig. 2 shows a simple truss, in which the joints a, c, e, g, i, and k comprise one group, and joints b, d, f, h, j, and l form the other.

Fig. 3 shows a compound truss, in which the sub-joints p, q, r, s, t, u, v, w, x, y, and z are a third group.

By the slope-deflexion method, the joint-rotations of the second group are computed by assuming those of the first group. The joint-rotations of the first group are computed by substituting those of the second group, and the procedure is then repeated.

Fig. 3



By the moment distribution method, the assumed moments are carried over from the joints of the first group to those of the second. The total unbalanced moments at the joints of the second group are distributed. Half the distributed moments are carried over from the joints of the second group to those of the first group, and the total unbalanced moments at the joints of the first group are distributed. The assumed carried-over moments from the joints of the first group to those of the second group are corrected, and the procedure again repeated.

EXAMPLE

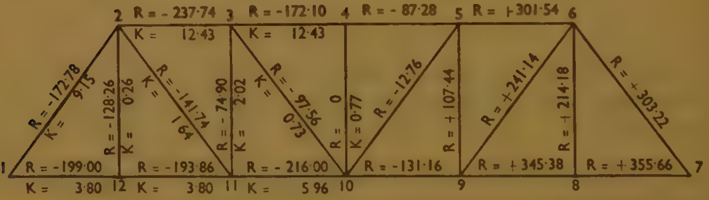
Slope Deflexion Method

Consider a six-panel Pratt truss with the stiffness factors and $2E\times$ rotations of the members as shown in *Fig. 4*. The slope-deflexion method is used to compute the secondary end moments in the members. The joints are divided into two groups composed of the odd- and even-numbered joints respectively. The assumed values for the joint rotations of the first group are as follows:

$$\theta_1 = -170 = R_{12}$$
$$\theta_3 = -200 = \tfrac{1}{2}(R_{23} + R_{34}) = \tfrac{1}{2}(-230 - 170)$$

Similarly, $\theta_5 = 110$; $\theta_7 = 300$; $\theta_9 = 100$; and $\theta_{11} = -200$.

Fig. 4



The simultaneous equations for the joint rotations of the second group are as follows:

$$9\cdot15\theta_1 + 0\cdot26\theta_{12} + 46\cdot96\theta_2 + 1\cdot64\theta_{11} + 12\cdot43\theta_3 = -14{,}404 \quad . \quad (2)$$
$$12\cdot43\theta_3 + 51\cdot26\theta_4 + 0\cdot77\theta_{10} + 12\cdot43\theta_5 = -9{,}673 \quad . \quad (4)$$
$$12\cdot43\theta_5 + 1\cdot64\theta_9 + 46\cdot96\theta_6 + 0\cdot26\theta_8 + 9\cdot15\theta_7 = 20{,}918 \quad . \quad (6)$$
$$3\cdot80\theta_9 + 0\cdot26\theta_6 + 15\cdot72\theta_8 + 3\cdot80\theta_7 = 8{,}156 \quad . \quad (8)$$
$$5\cdot96\theta_{11} + 0\cdot73\theta_3 + 0\cdot77\theta_4 + 28\cdot20\theta_{10} + 0\cdot73\theta_5 + 5\cdot96\theta_9 = -6{,}450 \quad . \quad (10)$$
$$3\cdot80\theta_1 + 15\cdot72\theta_{12} + 0\cdot26\theta_2 + 3\cdot80\theta_{11} = -4{,}577 \quad . \quad (12)$$

Equations (2), (4), and (6), can be solved by substituting the values of the θ's assumed above and by assuming also that: $\theta_{12} = -190$; $\theta_{10} = -170$; and $\theta_8 = 340$.

Hence, $\theta_2 = -212\cdot6$

$$= \frac{-14{,}404 - 9\cdot15(-170) - 0\cdot26(-190) - 1\cdot64(-200) - 12\cdot43(-200)}{46\cdot96}$$

Similarly, $\theta_4 = -164\cdot3$ and $\theta_6 = 352\cdot6$.

Equations (8), (10), and (12) can be solved by substituting the values of the θ's assumed and calculated above.

Therefore, $\theta_8 = 416\cdot3 = \dfrac{8{,}156 - 3\cdot80 \times 100 - 0\cdot26 \times 352\cdot6 - 3\cdot80 \times 300}{15\cdot72}$

Similarly, $\theta_{10} = -200\cdot0$ and $\theta_{12} = -198\cdot3$

The simultaneous equations for the joint rotations of the first group are as follows :

$$12{\cdot}43\theta_2 + 2{\cdot}02\theta_{11} + 55{\cdot}22\theta_3 + 12{\cdot}43\theta_4 + 0{\cdot}73\theta_{10} = -\ 15{,}952 \quad . \quad . \quad . \quad (3)$$

$$12{\cdot}43\theta_4 + 0{\cdot}73\theta_{10} + 55{\cdot}22\theta_5 + 2{\cdot}02\theta_9 + 12{\cdot}43\theta_6 = 8{,}612 \quad . \quad . \quad . \quad (5)$$

$$9{\cdot}15\theta_6 + 3{\cdot}80\theta_8 + 25{\cdot}90\theta_7 = 12{,}378 \quad . \quad . \quad . \quad (7)$$

$$5{\cdot}96\theta_{10} + 2{\cdot}02\theta_5 + 26{\cdot}84\theta_9 + 1{\cdot}64\theta_6 + 3{\cdot}80\theta_8 = 3{,}429 \quad . \quad . \quad . \quad (9)$$

$$3{\cdot}80\theta_{12} + 1{\cdot}64\theta_2 + 26{\cdot}84\theta_{11} + 2{\cdot}02\theta_3 + 5{\cdot}96\theta_{10} = -\ 7{,}223 \quad . \quad . \quad . \quad (11)$$

$$25{\cdot}90\theta_1 + 3{\cdot}80\theta_{12} + 9{\cdot}15\theta_2 = -\ 7{,}012 \quad . \quad . \quad . \quad (1)$$

Equations (3) and (5) can be solved in the same manner as equations (8), (10), and (12). Thus :

$$\theta_3 = -\ 194{\cdot}1$$

$$= \frac{-15{,}952-12{\cdot}43(-212{\cdot}6)-2{\cdot}02(-200)-12{\cdot}43(-164{\cdot}3)-0{\cdot}73(-200{\cdot}0)}{55{\cdot}22}$$

Similarly, $\theta_5 = 112{\cdot}6$

Equations (7), (9), (11), and (1) can be solved by substituting all the values of θ calculated above.

$$\text{Thus,} \quad \theta_7 = \frac{292{\cdot}3 - 12{,}378 - 9{\cdot}15 \times 352{\cdot}6 - 3{\cdot}80 \times 416{\cdot}3}{25{\cdot}90}$$

Similarly, $\theta_9 = 83{\cdot}2$; $\theta_{11} = -\ 169{\cdot}0$; and $\theta_1 = -\ 166{\cdot}5$.

The first approximations are thus completed. The first corrections and the final values of all the θ's are given in Table 1.

TABLE 1

Order of Solution	Group	Unknowns	First Approximation	First Correction	Final Value
1	2nd	θ_2	− 212·6	− 3·3	− 215·9(− 216·8)*
2		θ_4	− 164·3	− 1·6	− 165·9(− 166·1)
3		θ_6	352·6	1·0	353·6 (353·6)
4		θ_8	416·3	5·9	422·2 (421·4)
5		θ_{10}	− 200·0	− 3·2	− 203·2(− 203·8)
6		θ_{12}	− 198·3	− 8·3	− 206·6(− 206·8)
7	1st	θ_3	− 194·1	0·0	− 194·1(− 194·4)
8		θ_5	112·6	0·8	113·4 (113·5)
9		θ_7	292·3	− 1·2	291·1 (290·6)
10		θ_9	83·2	− 0·3	82·9 (83·1)
11		θ_{11}	− 169·0	2·1	− 166·9(− 167·0)
12		θ_1	− 166·5	2·4	− 164·1(− 163·8)

The secondary end moments in all the members are calculated as follows :

$$M_{1-2} = -\ 235\ (-\ 236)^{*} \text{ inch-lb.}$$

$$= \quad 9{\cdot}15\{2(-\ 164{\cdot}1) + (-\ 215{\cdot}9) - 3(-\ 172{\cdot}8)\}$$

---

* The values in parentheses are the "more exact" values given by Johnson, Bryan, and Turneaure (see reference 3, p. 104).

$M_{7-6} = \quad 240\ (234)$

Similarly,

$M_{1-12} = \quad 236\ (235)$

$M_{7-8} = -239\ (-236)$

$M_{2-1} = -731\ (-717) = 9{\cdot}15\{2(-215{\cdot}9) + (-166{\cdot}5) - 3(-172{\cdot}8)\}$

$M_{6-7} = \quad 823\ (807)$

Similarly,

$M_{2-12} = -65\ (-65)$
$M_{2-11} = -288\ (-286)$
$M_{2-3} = 1{,}084\ (1{,}071)$
$M_{3-2} = 1{,}355\ (1{,}352)$
$M_{3-11} = -667\ (-666)$
$M_{3\ 10} = -218\ (-218)$
$M_{3-4} = -470\ (-467)$
$M_{4-3} = -119\ (-124)$
$M_{4-10} = -414\ (-415)$
$M_{8-7} = 265\ (266)$
$M_{8-6} = 142\ (142)$
$M_{8-9} = -413\ (-407)$
$M_{9-8} = -1{,}703 (-1{,}700)$
$M_{9-6} = -334\ (-335)$
$M_{9-5} = -87\ (-87)$
$M_{9-10} = 2{,}123\ (2{,}122)$
$M_{10-9} = 420\ (415)$
$M_{10-5} = -187\ (-186)$
$M_{10-4} = -443\ (-445)$

$M_{6-8} = 124\ (124)$
$M_{6-9} = 110\ (109)$
$M_{6-5} = -1{,}053 (-1{,}044)$
$M_{5-6} = -4{,}029 (-4{,}027)$
$M_{5-9} = -25\ (-25)$
$M_{5-10} = 45\ (45)$
$M_{5-4} = 4{,}012\ (4{,}012)$
$M_{4-5} = 531\ (534)$

$M_{12-1} = 66\ (67)$
$M_{12-2} = -62\ (-63)$
$M_{12-11} = -2\ (-3)$
$M_{11-12} = 157\ (153)$
$M_{11-2} = -204\ (-205)$
$M_{11-3} = -612\ (-612)$
$M_{11-10} = 662\ (663)$
$M_{10-11} = 433\ (443)$
$M_{10-3} = -225\ (-225)$

For the moments around the joints of the second group, it is preferable to use the first approximations for the joint rotations at the far ends of the members meeting at the joint, instead of the final values.

Moment Distribution Method

In this method the joints are again divided into two groups as in the slope-deflexion method, and the fixed-end moments around all the joints are computed as follows:—

Joint 2, member 2–1, fixed-end moment $= -3 \times 9{\cdot}15(-172{\cdot}8)$
$= 4743$ inch-lb.

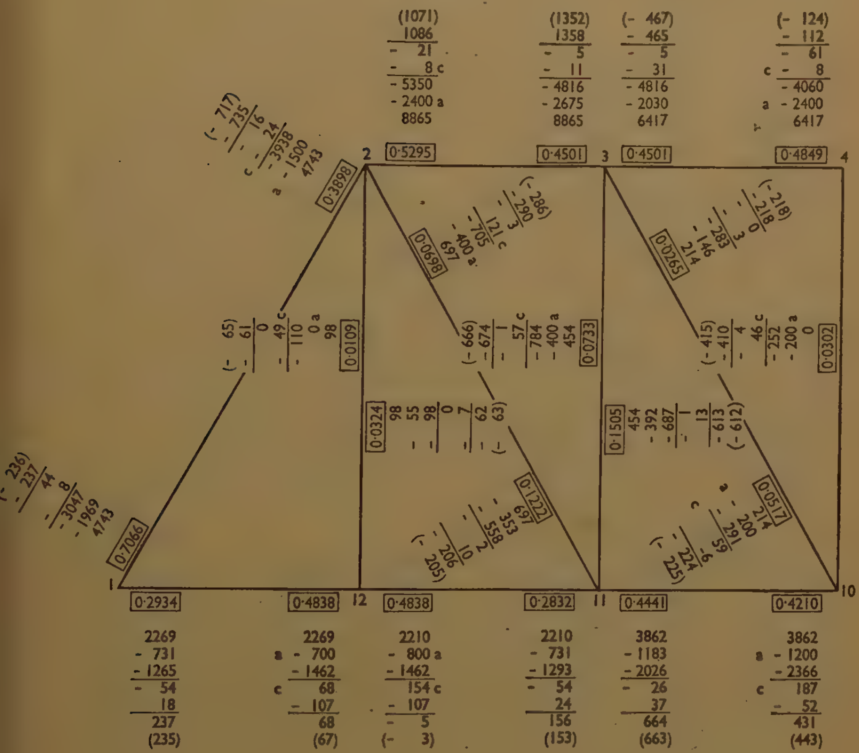
The assumed carried-over moments from the joints of the first group to those of the second group are calculated as follows:—

Joint 2, member 2–1 assumed carried over moment $= 9(-170)$
$= -1{,}500$ inch-lb.
„ 2, „ 2–12 „ $= 0(-190)$
$= 0$ inch-lb.

Joint 2, member 2–11 assumed carried over moment = 2(— 200)
= — 400 inch-lb.
" 2, " 2–3 " = 12(— 200)
= —2,400 inch-bl.

For explicitness the symbol a is written after the assumed carried-over moments on the calculation sheet (see *Figs 5*).

Fig. 5 (a)

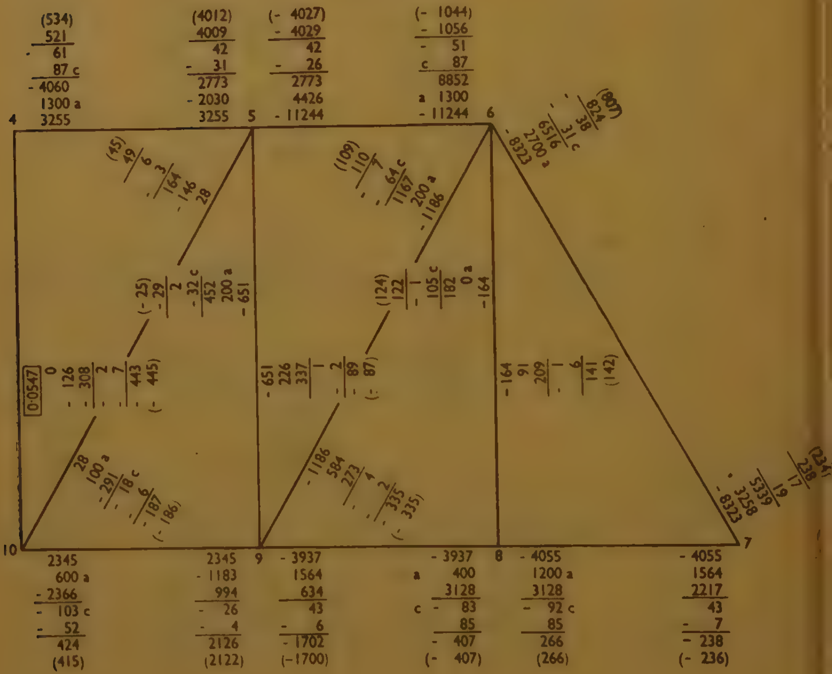


The total unbalanced moments at the joints of the second group are distributed, and half of the distributed moments are carried over from the joints of the second group to those of the first group. The total unbalanced moments at the joints of the first group are then distributed in the same way as in the conventional method, and the assumed carried-over moments from the joints of the first group to those of the second group are corrected. The corrected carried-over moments of the joints of the second group are completed as follows :—

Joint 2, member 2–1 corrected carried-over moment
$= (-3{,}047)/2 + 1{,}500 = -24$
„ 2, „ 2–12, „ $= (-98)/2 + 0 = -49$
„ 2, „ 2–11, „ $= (-558)/2 + 400 = 121$
„ 2, „ 2–3, „ $= (-4{,}816)/2 + 2{,}400 = -8$
etc.

For explicitness, the symbol c is written after the corrected carried-over moments in the calculation sheet (see *Figs 5*).

Fig. 5 (b)

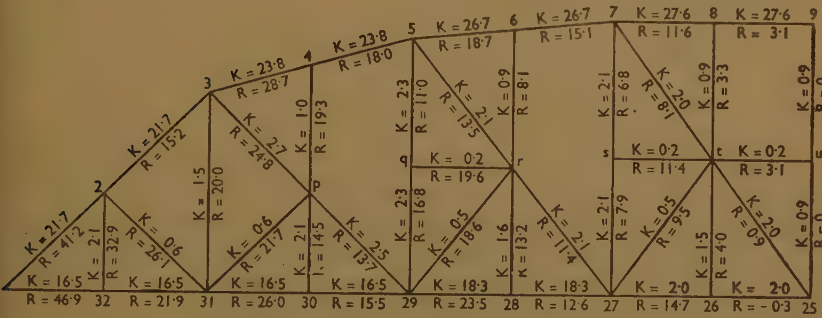


The total unbalanced moments at the joints of the second group are again distributed, and half of the distributed moments are carried over from the joints of the second group to those of the first group. The total unbalanced moments at the joints of the first group are distributed, as in the conventional method. The two cycles of approximations are now completed. All the moments, including the fixed-end, the carried-over (assumed, corrected, and calculated) and the distributed moments are added together as in the conventional method.

FURTHER EXAMPLE—KENOVA BRIDGE

Fig. 6 refers to the Kenova bridge under full load and shows a sixteen-panel Petit truss. The values of K (stiffness factor) and R ($2E \times$ rotation of the member) are given for each member. In order to estimate the joint rotations, the joints are divided into three groups. The odd- and even-numbered joints form the first and second groups, and the remaining joints compose the third group. The joint rotations are estimated as follows:

Fig. 6



$\theta_1 = 44(51{\cdot}3)^* = (41 + 47)/2$

$\theta_5 = 18(16{\cdot}6) = (18 + 19)/2$

$\theta_9 = 0$, by symmetry

$\theta_{27} = 13(10{\cdot}0) = 13/1$

$\theta_{31} = 24(22{\cdot}4) = (22 + 26)/2$

$\theta_3 = 22(21{\cdot}2) = (15 + 29)/2$

$\theta_7 = 13(13{\cdot}3) = (15 + 12)/2$

$\theta_{25} = 0$, by symmetry

$\theta_{29} = 20(19{\cdot}1) = (16 + 24)/2$

$\theta_2 = 28(24{\cdot}7) = (41 + 15)/2$

$\theta_6 = 17(17{\cdot}7) = (19 + 15)/2$

$\theta_{26} = 6\ (7{\cdot}8) = (15 + 4 + 0)/3$

$\theta_{30} = 21(20{\cdot}3) = (26 + 16)/2$

$\theta_p = 18(17{\cdot}2) = (25 + 15 + 14)/3$

$\theta_r = 13(11{\cdot}3) = (14 + 13 + 11)/3$

$\theta_t = 4\ (3{\cdot}7) = (8 + 4 + 1)/3$

$\theta_4 = 23(25{\cdot}4) = (29 + 18)/2$

$\theta_8 = 7\ (7{\cdot}6) = (12 + 3)/2$

$\theta_{28} = 18(19{\cdot}6) = (24 + 13)/2$

$\theta_{32} = 34(33{\cdot}5) = (47 + 22)/2$

$\theta_q = 14(12{\cdot}6) = (11 + 17)/2$

$\theta_s = 7\ (5{\cdot}7) = (7 + 8)/2$

$\theta_u = 0$, by symmetry

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* The values in parentheses are the final values given by Parcel and Maney (see reference 4).

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3. Johnson, Bryan, and Turneaure, "Theory and Practice of Modern Framed Structures," part 2 (1929).
4. J. I. Parcel and G. A. Maney, "Elementary Treatise on Statically Indeterminate Stresses," New York, 1936.

The Paper is accompanied by four sheets of diagrams, from which the Figures in the text have been prepared.

Paper No. 5828

"Scale-Model Experiments in Connexion with the Design of a Large Lock"

by

John Allen, M.Sc., Ph.D., and Basil Standing, M.Sc., A.M.I.C.E.

(Ordered by the Council to be published in abstract form) †

Previous Researches on Model-Locks

The first experiments were apparently carried out by Krey[1] in 1911 (quoted by Jansson[2]) followed by others at Karlsruhe[3] in 1923, and in 1924 by an extensive series[4] in the Experimental Institute in Berlin. Since then numerous other experiments have been performed, for example, by Cabelka,[5] and particularly by the Tennessee Valley Authority,[6] whilst Griffin,[7] in a highly interesting Paper, has discussed the conditions peculiar to a number of prototypes, and the bearing of the model-tests on the subsequent design.

The Present Problem

A large lock is under construction at Eastham, Cheshire, primarily for the berthing of oil tankers of approximately 28,000 tons gross register—a size considerably in excess of any tanker at present entering British ports. To keep the level in the dock-basin at a reasonably constant height, it is necessary to construct an entrance lock leading from the River Mersey into the dock. The length of the lock will be 807 feet and the breadth 100 feet. The siting of the lock and dock in relation to the existing locks at Eastham is shown in *Fig. 1.*

Figs 2 show the general dimensions and the arrangement of the filling and emptying system of the lock. In order to raise a ship entering the lock from the River Mersey, it is planned to open the valves in the two side culverts, the water entering the lock by means of sluices connecting the lock with the culverts. For emptying the lock into the river, another set of sluices and valves is to be utilized.

The Manchester Ship Canal Company, for whom the dock is being constructed, were anxious that the filling or emptying of the lock should not cause dangerously large movements of the ship, and that any silt deposited on the floor of the lock should not reduce either the maximum available lift in the lock or the size of vessels which could use the lock.

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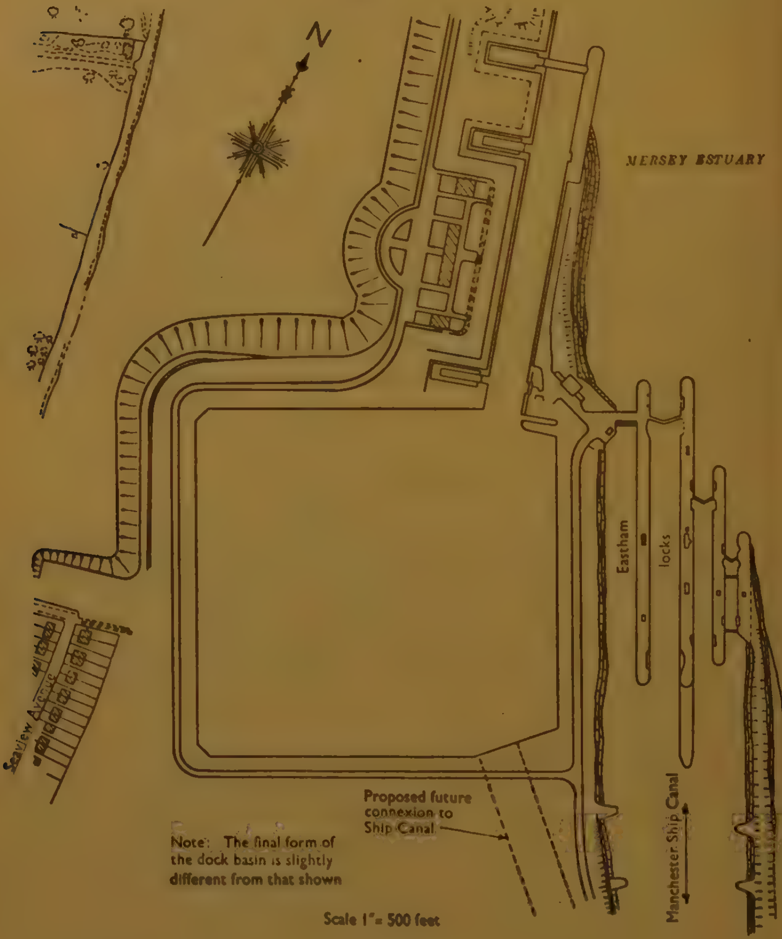
[1] The references are given on p. 112.

† The full MS. and illustrations may be seen in the Institution Library.—Sec. I.C.E.

Accordingly a $\frac{1}{48}$-scale model of the lock was constructed at Manchester University, to test various designs of sluices, and to record the ensuing motion of a model-tanker placed in the lock.

Fig. 1



Note: The final form of the dock basin is slightly different from that shown

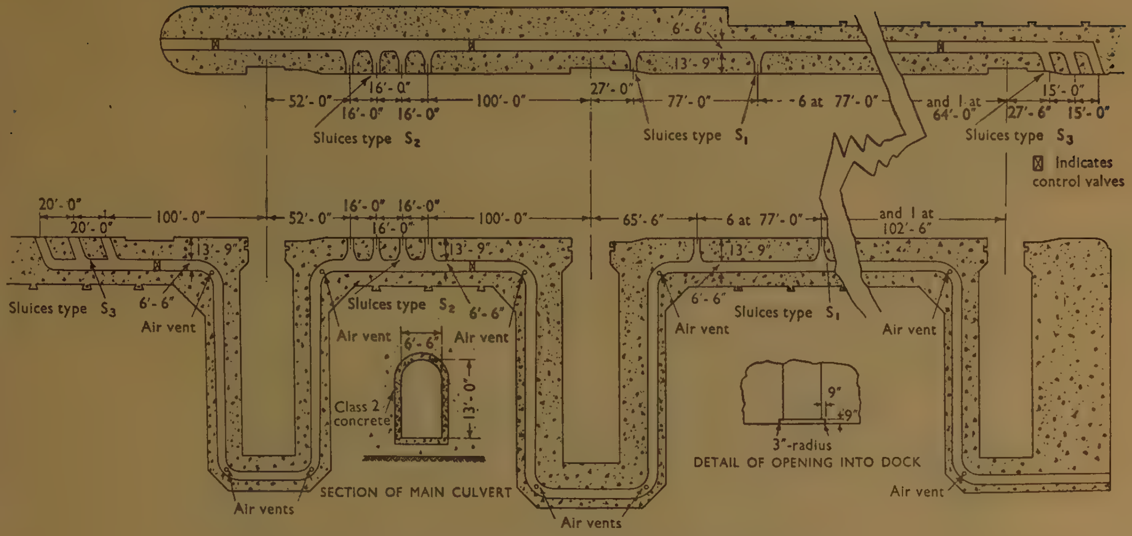
Scale 1" = 500 feet

Site Plan

Description of the Model

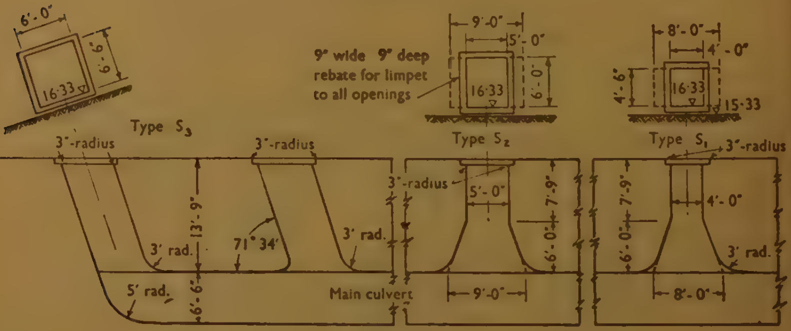
The model lock was constructed of timber and Swedish hard-board (" Masonite "). The scale-model tanker was carved from wood, whilst

Figs 2 (a)



SECTIONAL PLAN OF LOCK ON CENTRE-LINE OF CULVERT the sluices were cast from plaster of Paris. Sheet-metal was used for the side culverts.

Figs 2 (b)



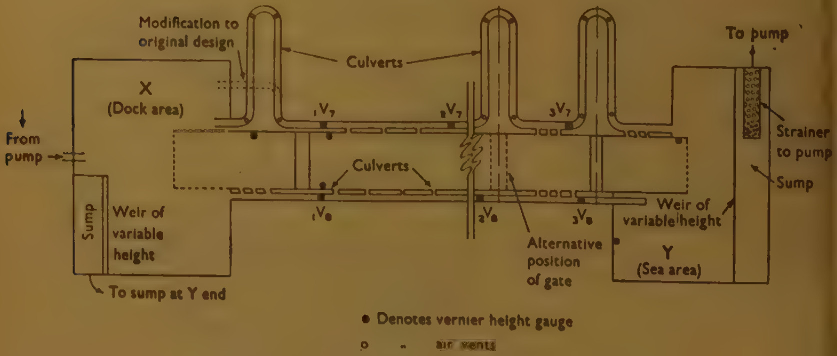
Scale: 1 inch = 25 feet

Culvert Details

The lay-out of the model is shown in *Figs 3* and *4*.

In the Paper, the Authors proceed to calculate the scales of velocity, time, discharge, etc.

Fig. 3

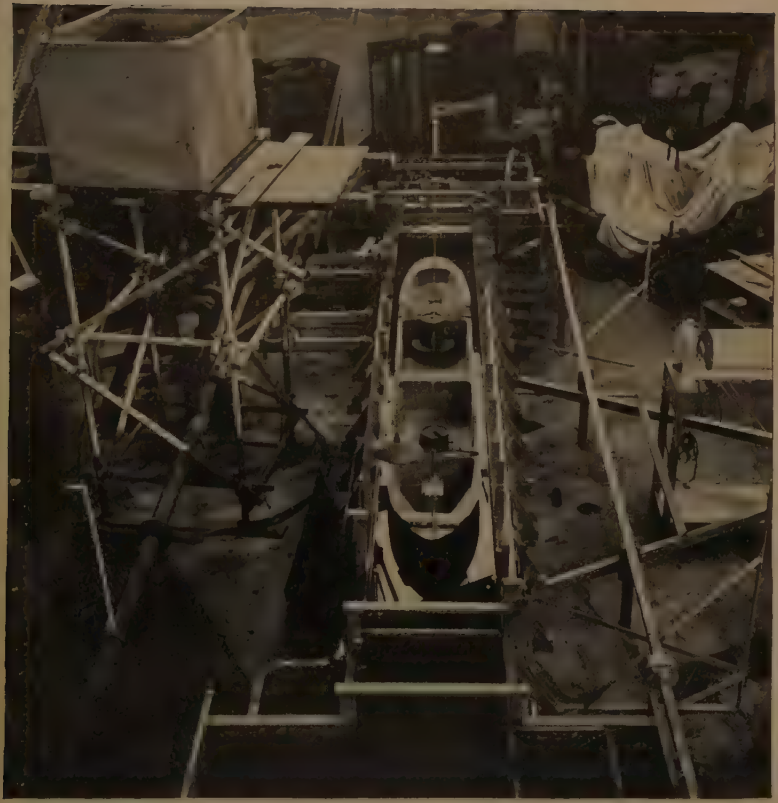


Arrangement of Apparatus (Diagrammatic)

The Recording Apparatus

Since a permanent record of the movements of the vessel in several directions was required it was decided to use a mechanically operated recorder, rather than a system incorporating cathode-ray oscillographs or

Fig. 4



General View of Model, Looking Downstream

other instantaneous recording devices. The Paper describes, with photographs, how a chart-type of recorder was arranged, so that the movements of the ship during a locking operation could be traced upon a single sheet of graduated paper wrapped around a rotating cylinder (*Fig. 5*). To transmit the motions of the ship to the recording pens, a system of wires and guide pulleys was evolved. The movements of the vessels were subsequently converted to those which might be expected in the full-size ship, and have been given in tabular form.,

EXPERIMENTAL PROCEDURE

The sluices were moulded in the model, and tested by recording the motions of the boat corresponding to various values of the lift in the lock. Experiments were also performed with the vessel lowered, that is to say, with the lock partially emptied. In addition, the draught of the model was varied to correspond to ballasted and fully loaded conditions.

During each locking operation the time was recorded, firstly by means of a stop-watch, and secondly by noting the number of graduations through which the pens had travelled upon the recorder chart. Since the time taken by the pens to traverse the distance between successive graduations was two seconds, the time of locking could easily be calculated and compared with that given by the watch.

For each lift, the locking was performed using (1) each side culvert in turn, and (2) both culverts simultaneously, so that all possible conditions were tried, and the resulting motions of the vessel examined.

TESTS TO DETERMINE THE EFFECTS OF SILTATION

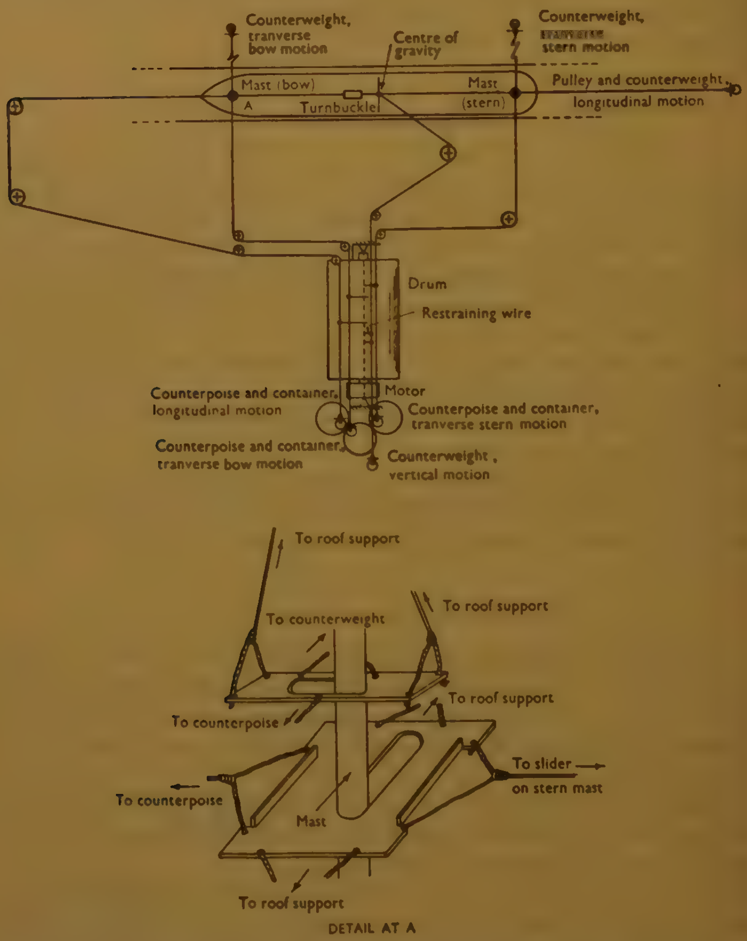
A flat bed of powdered pumice was laid in the lock, and photographs taken after 13, 26 and 39 lockings under the highest possible lift of 25·5 feet in nature. These experiments, which were undertaken both with and without the tanker in the lock, were repeated for two different types of sluices.

Since the photographs taken after 39 lockings showed that the bed configuration was about the same as after 13 and 26 lockings, it was inferred that a state of stability had been reached.

Although the laying of a flat bed of pumice might appear to be an artificial way of attacking the problem of siltation, it was the best solution which could be found without including a tidal portion of the River Mersey in the model. In the prototype, silt would be carried in suspension by the river, and, when filling on the tide, the water entering the lock would bring in a quantity of material which would be deposited as a more or less even bed, on the floor of the lock.

Successive fillings and emptyings would then scour the bed in the vicinity of the ports, whilst the material brought in, when filling the lock from the river would tend to increase the depth of the bed. Thus, exact information as to silt movements on the floor of the lock, could only have been obtained by reproducing the cycles of operation likely to occur in

Figs 5


Counterweight, tranverse bow motion
Centre of gravity
Counterweight, tranverse stern motion
Mast (bow)
A
Turnbuckle
Mast (stern)
Pulley and counterweight, longitudinal motion
Drum
Restraining wire
Counterpoise and container, longitudinal motion
Motor
Counterpoise and container, tranverse stern motion
Counterpoise and container, tranverse bow motion
Counterweight, vertical motion
To roof support
To counterweight
To roof support
To roof support
To counterpoise
To slider on stern mast
To counterpoise
Mast
To roof support
DETAIL AT A

Diagrammatic Representation of Force-Recording System (above) and Detail at Bow Mast (below)

practice, at the same time investigating the criteria for dynamical similarity with regard to silt movement. Considerations of space and time precluded all possibility of attempting this ideal method.

RESULTS AND CONCLUSIONS

The results showed that both types of sluices used for filling the lock would be satisfactory in that, (*a*) the maximum calculated forces required to arrest the motion of the ship were not excessive when due consideration was given to the size of the vessel and lock, and (*b*), there seemed to be no likelihood of any blockage of the lock or sluices through heavy siltation.

Since one type of port would, however, be easier to "blank-off" than the other, it was decided to recommend that design on the grounds of easier maintenance.

APPENDICES

Appendix 1 of the Paper provides an estimate of the effect of mutual interference between the systems of recording the transverse and longitudinal movements of the vessel, thereby justifying the use of slotted plates as connexions from the recorder-wires to the masts.

Appendix 2 describes subsidiary experiments performed on a small lock, which was used in order that only one sluice needed to be moulded and tested for its effect on the siltation. Thus time was saved by avoiding the moulding of fifteen such sluices in the large model-lock—a task occupying almost a week.

Only those designs of sluices which gave encouraging results when tested in the small lock were afterwards moulded in the large lock.

Appendix 3 provides examples of the calculations of the velocities through the various sluices, and sections of the culverts, assuming certain values for the frictional factors of the culverts, and for the losses at the sluices, etc.

The times of locking have also been computed, and are in reasonable agreement with those observed.

The interesting point emerges that, when filling the lock from the dock, the sluices furthest downstream in the culverts give a greater rate of discharge than those near to the dock. This is in accordance with the results of the research performed by the Tennessee Valley Authority and agrees with observations taken on the existing locks at the beginning of the Ship Canal at Eastham.

It is hoped to make further comparisons of the theoretical and actual filling times when the new lock goes into service.

ACKNOWLEDGEMENTS

The Authors wish to record their gratitude to Mr C. M. Mason, T.D., M.Sc., A.M.I.C.E., for the facilities granted in the Whitworth Engineering Laboratories, Manchester University, and to Professor A. H. Gibson, D.Sc., LL.D., M.I.C.E., for his continued encouragement and advice.

They are also indebted to Mr D. C. Milne, B.Sc., M.I.C.E., Chief Engineer to the Manchester Ship Canal Co., and to his staff, particularly Mr A. E. Blyth, B.Sc., A.M.I.C.E., who supervised the construction of the model.

In addition, thanks are due to Mr E. G. Gresty, B.Sc.Tech., of the Lea Recorder Company for his assistance in the design and construction of the recording apparatus.

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6. "Hydraulic Model Investigation of Lock Filling and Emptying Systems." Tennesee Valley Authority, Technical Monograph No. 64, 1947.
7. A. F. Griffin, "Influence of Model Testing on Lock Design." Proc. Amer. Soc. Civ. Engrs., vol. 76, No. 1, Part 5 (Jan. 1950).

The Paper is accompanied by twenty-six photographs and thirteen diagrams, from some of which the half-tone page plate and the Figures in the text have been prepared.

OBITUARY

SIR THOMAS PEIRSON FRANK, who died in London on the 12th November, was born at Kirby Moorside in Yorkshire on the 23rd July 1881. Educated at Pickering Grammar School and Huddersfield Technical College, he had 3 years' practical training under Mr K. P. Campbell, then Borough Engineer of Huddersfield, and in 1903 was made Chief Assistant to the Borough Engineer of Dewsbury and Engineer to the Dewsbury and Heckmondwike Waterworks Board. There, for 5 years, he gained further experience of general construction and maintenance of roads and tramways, and was concerned with the building of a covered market and the laying of water mains and sewers.

In 1908 he secured his first independent post as City Engineer of Ripon, where he worked for 4 years and was responsible for the building of two river dams and for the extension of sewage disposal works. From Ripon he moved to Newark-on-Trent, where he was married in 1914 to Miss Irene Augusta Thirlway, and then to Stockton-on-Tees, Plymouth, Cardiff, and Liverpool in succession with his increasing experience and ability, holding posts as City or Borough Engineer for periods of 3 to 4 years in each place.

He was temporarily released from his post at Stockton-on-Tees to join the Royal Engineers in 1915, was commissioned in February 1916, and was posted to the 237th Field Company in France. In April 1917 he was given command of the 198th Land Drainage Company but was wounded in August of that year. After leaving hospital he was a lecturer at a School of Instruction until demobilized with the rank of Captain in 1918.

While at Plymouth he was responsible for construction work including roads and sewers for an extensive housing scheme, and a reinforced concrete landing stage. At Cardiff he acted as Surveyor to the East Glamorgan Town Planning Advisory Committee, and did the same for the South West Lancashire Committee while at Liverpool, where he also lectured at the University.

In 1930 he was selected for the post of City Engineer and County Surveyor to the London County Council, and in the next 9 years works carried out under his direction included improvements in main drainage and sewage installations, housing estate developments, the widening of Putney bridge, the building of a new bridge at Wandsworth, and the demolition of the old Waterloo bridge and the construction of the new one in co-operation with consulting engineers. He was a member of the Experimental Roads Committee of the Ministry of Transport, of the

Highways Research Board of the Department of Scientific and Industrial Research, and the Departmental Committee on Garden Cities and Satellite Towns.

Throughout the 1939–45 war he was Co-ordinating Officer for Road Repairs and Public Utility Services for the London area, and directed the repair services that enabled London to carry on in spite of the severest air raids. In recognition of his work he was knighted in 1942.

In July 1946, at the age of 65, he retired from the London County Council and a month later joined, as a partner, the Westminster firm of Coode, Vaughan-Lee and Gwyther, consulting engineers; and undertook a variety of work including reports on sea defence works at Chesil Bank, a flood prevention scheme for Salford, and water storage for irrigation in British Guiana. He directed the designing of the Cross River, Gongola, Garafini, and Foggo bridges in Nigeria, and was concerned with work on power stations in Malaya and Melbourne, and a tunnel being driven under the Thames at Long Ditton.

Sir Peirson Frank's position and ability resulted in his being asked to officiate within numerous national and engineering Institutions. He was President of the Institution of Civil Engineers from 1945 to 1946, having been elected an Associate Member in 1906, transferred to full Membership in 1922, and become a Member of Council in 1931. He was also President of the Town Planning Institute in 1944, a Member of Council of the Royal Institution of Chartered Surveyors, a Fellow of the Royal Sanitary Institute, and a Member of the Institution of Municipal Engineers. In 1941 he become a Member of the Executive Committee of the Automobile Association and in November 1950, Vice-Chairman. During the summer of 1950 he took part in a tour of Canada and the United States to study road conditions and reciprocal services between the A.A. and its trans-atlantic counterparts. He was a Colonel in the Engineer and Railway Staff Corps from 1932 onwards.

He is survived by his wife, one son, and four daughters.

DAVID BARRINGTON BROW, O.B.E., M.C., who died in Bermuda on the 27th July 1951 at the age of 60, was born on the 14th August 1891 at Dartford, Kent, where his father David F. Brow was at one time Principal of the Technical College. Educated at Dartford Grammar School and privately, he was articled to the late William Banks, a civil engineer, for 3½ years, and then in 1912 was made assistant engineer on the construction of the Rochester and Southwark bridges. In 1914 he gained further experience as an assistant, on the Doncaster–Thorne widening of the Great Central Railway, before the war began and he joined the army. He served with the West Kent Yeomanry in Gallipoli and with the Royal Engineers in France, won the M.C., and ended his full-time army service as a garrison engineer on the N.W. frontier of India, with the rank of Major.

Demobilized in 1921, he was appointed Executive Engineer to Karachi Municipal Corporation, and 2 years later Engineer to the Port Trust of that city. Considerable development had been planned for the Port, and he took charge of the construction of the new West Wharf, a deep-water quay 2,500 feet long and founded on monoliths. The first two berths were finished in 1927, and two more in 1929. He was also in charge of extensive reclamation schemes, and for 3 years was responsible for the harbour dredging.

In 1936 he was appointed Deputy Chief Engineer to the Port Trust, and in February 1937 Chief Engineer; and that post he held throughout the difficult years of the 1939–45 war. With the entry of Japan and America into the war the Port became very busy, and the quantity of cargo handled rose to a daily average, in 1944–45, of 7,800 tons; to deal with this traffic a total of 900 feet of new wharfage and a road along the East Wharves were built. Facilities were also provided for the marine engineers to repair large numbers of merchant and naval ships. In 1946 he became Chairman of the Trust for 1 year, before retiring to take up consulting work in Pakistan. He was ill when he retired, and continuing poor health resulted in his leaving Karachi for Bermuda in December 1950.

D. B. Brow was elected an Associate Member of the Institution in January 1919, a Member in March 1937, and represented India on the Council from November 1945 to November 1948. His services during the 1939–45 war were recognized by the award of the O.B.E.

CORRIGENDA

Journal November 1951, Paper No. 5820 (Carroll)—
p. 494, line 2, *for* " square " *read* " linear "
line 3, *for* " slab " *read* " beam "
line 11, *for* " $x = 2{,}148$ per square foot "
read " $x = 2{,}148$ per linear foot "

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